OPTIMAL PLANNING OF RUNNING TIME IMPROVEMENTS FOR MIXED-USE FREIGHT AND PASSENGER RAILWAY LINES

BY

BRENNAN MICHAEL CAUGHRON

THESIS

Submitted in partial fulfillment of the requirements
for the degree of Master of Science in Civil Engineering
in the Graduate College of the
University of Illinois at Urbana-Champaign, 2017

Urbana, Illinois

Advisers:

Professor Christopher P.L. Barkan, Adviser
Mr. C. Tyler Dick, Co-Adviser

ABSTRACT

In recent years, the United States has seen a renewed focus on developing improved intercity passenger railway lines and services. With the political sensitivity of public investment in rail infrastructure and accompanying shortage of state and federal funds, it is important that the most cost effective investments are selected. Many of these endeavors, including high-speed-rail projects with new, dedicated segments, involve infrastructure investments targeted at improving the speed, capacity, and reliability of existing railway lines. In most cases, these existing lines support the operation of commingled passenger and freight traffic on the same trackage. These shared trackage arrangements introduce numerous engineering and operating challenges to successfully planning and executing improvement projects. Freight, commuter, and intercity rail traffic types have inherently different performance and service characteristics that further complicate the planning of infrastructure improvements. This thesis is focused on enhancing the planning methodology of intercity passenger rail service in the United States.

Chapter 1: Background of American Intercity Passenger Rail

The past decade has seen substantial increases in ridership and revenue for Amtrak services. Even with the increases outlined in this chapter, there remain differences in the level of rail service between different regional corridors. Increased passenger service speeds and frequencies are matters of concern to class 1 freight railroads. Passenger service improvements must be planned in a manner that preserves the safety of the passengers as well as present and future franchise of freight carriers. Organizations sponsoring new or improved rail service typically commission feasibility studies, conducted in partnership with Amtrak, to analyze the costs and benefits of changes to intercity passenger service. Service improvements should be executed within the context of a long-range strategy for passenger rail.

Chapter 2: A Spreadsheet Based Train Performance Calculator

A train performance calculator (TPC) is a computer program used to determine the running time of a train over a route and requires input parameters related to the physical and performance characteristics of route and train consist. Using this information, the software will compute a best-case speed versus distance performance of a train. This chapter describes a TPC based in Microsoft Excel. Through the use of Visual Basic for Applications scripts, the running time of passenger or freight trains can be computed in several minutes of processing time. Using this approach, agencies charged with planning improvements to rail corridors can perform relative assessments of different infrastructure, rolling stock, and train operating scenarios. The TPC described in this chapter offers a rapid and reasonably accurate methodology to provide the needed results using an inexpensive and widely available spreadsheet software.

Chapter 3: Increasing Passenger Train Speeds through Curve Realignment and Rolling Stock Improvements

In order to reduce running times on lines with civil speed restrictions due to curvature, selected curves may be re-aligned to increase the allowable speed and rolling stock may be introduced with better curving performance. For an existing curve, there is a maximum potential speed benefit that can be achieved given the right-of-way characteristics as well as engineering and operating constraints of the corridor. This chapter presents a literature review of topics related to higher train curving speed and outlines the relationships between existing conditions and the maximum potential curving speed. The results illustrate the difference in curving speed improvement projects for rail lines with predominantly freight traffic compared to lines that are mostly passenger. Using this research, planners and engineers of passenger rail systems can gain a better understanding of the speed improvement benefits that can be expected when upgrading an existing railway line and in turn determine the relative cost effectiveness of different improvement strategies.

Chapter 4: A Project Selection Model for Improving Running Time on Intercity Passenger Railway Lines

Several alternatives exist for reducing running time and increasing average passenger train speeds, including investments in track, signal, highway grade crossing, and rolling stock improvements. This chapter presents a methodology for optimally selecting projects or establishing infrastructure budgets to reduce running time on a passenger rail corridor. A mixed integer program is formulated to solve this problem and the model is applied to a case study route. The model input information includes capital improvement, track maintenance costs, existing route conditions, and rolling stock performance. This model can be used as part of a methodology for quickly and efficiently developing a strategic plan for improving minimum travel time on passenger rail corridors.

Chapter 5: Conclusions and Future Work

This chapter serves as a review of findings for the previous chapters and outlines several ideas for refinements and future work in this research area.

ACKNOWLEDGEMENTS

I would like to thank Paragon Decision Technology for granting use of the AIMMS software academic license. The AIMMS software proved instrumental in particular to developing and troubleshooting the optimization model presented in Chapter 4.

I am grateful to Michael Garcia (retired IDOT), David Staplin (retired Amtrak), and Samuel Sogin (Union Pacific) for technical insights regarding the topic of this paper. I would like to thank C. Tyler Dick of UIUC for sharing his railway engineering experience and expertise at a critical stage of the development of this work. Tyler provided numerous helpful insights in the technical aspects of railway design and was exceedingly patient with my delays in developing draft publications.

This research was conducted with the support of the National University Rail Center, a US DOT OST Tier 1 University Transportation Center and the Association of American Railroads. In addition, during a portion of this work the author was supported by grant BAA-2010-1 from the United States Federal Railroad Administration. All of this support was made possible by Dr. Christopher P.L Barkan of the University of Illinois. Chris served as my graduate advisor and was a great friend and mentor throughout both my undergraduate and graduate studies. I will forever be in debt to Chris for the tremendous opportunities he opened up to me.

While interning at the National Railroad Passenger Corporation, I experienced hands on the methods industry experts use to manage intercity passenger rail service. Walter Lander, William Pedroza, and Bruce Hilblom were excellent coworkers from whom I learned a great deal about Amtrak. I would also like to thank Richard Hoffman, with whom I worked closely on various passenger rail feasibility studies. Dick shared with me his management experience and lessons from a long railroad career at the Chicago and North Western Railway, various railcar manufacturing firms, and Amtrak. Finally, I would like to thank my supervisor at the time Michael Franke for sharing his knowledge and experience from a long career in both the freight and passenger sectors in the rail industry.

Many friends and family have taken a large role in developing and supporting my interest in the railway industry. My late grandfather Donald Becker provided the initial spark that has culminated in the career I am now embarking on. In addition, my uncles Joseph and Michael Becker were ever supportive in fostering my transportation interests. Finally, Jeffrey Hook served as a great friend and mentor and helped me develop an engineering thought methodology and mechanical intuition that served as the foundation for my engineering education and career.

I am most grateful to my parents Ann Marie and Michael. I feel that I only begin to understand their dedication and sacrifice in raising my sisters and me. Their loving support has been the cornerstone of this and every one of my pursuits.

Finally, I would like to acknowledge my wife, Dana, for providing the loving support, encouragement, and sometimes honest discipline necessary for me to make better efforts towards finishing the many unfinished projects in my life.

This work and any statements or conclusions herein do not represent the views of BNSF Railway Co. Any mention of Class 1 railroad policy is based on the authors own opinion and general understanding of industry trends and should not be taken in any way as an official statement or position.

To Dana

TABLE OF CONTENTS

CHAPTER 1: BACKGROUND ON AMERICAN INTERCITY PASSENGER RAIL

1.1 Recent Growth of American Intercity Passenger Rail

Amtrak passenger rail services are organized into three categories: long distance, state supported/short-distance, and Northeast Corridor services. Ridership on all three categories has grown substantially since the turn of the 21st century. Figures 1.1 and 1.2 show yearly ridership and revenue totals for each of the rail service categories. From 2000 to 2011, ridership grew 12.6% for long distance services, 29.8% for Northeast Corridor services, and 72.1% for short distance services. Over the same time period, ticket revenue has also grown 25.2% for long distance services, 86.9% for short distance services, and over 104% for Northeast Corridor services.

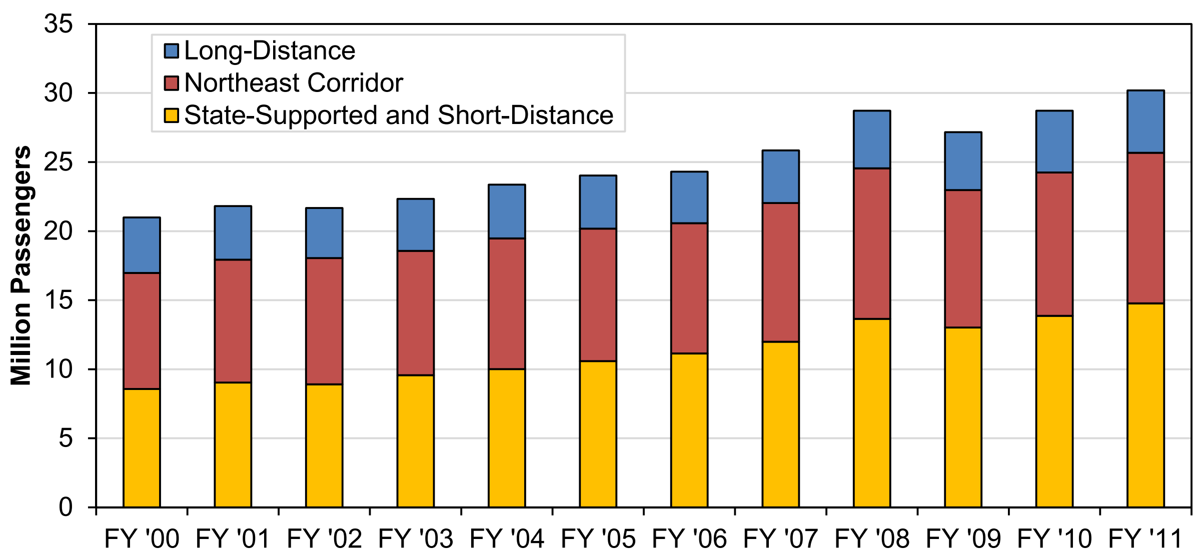


Figure 1.1: Amtrak Ridership Since Year 2000 (Amtrak, 2012)

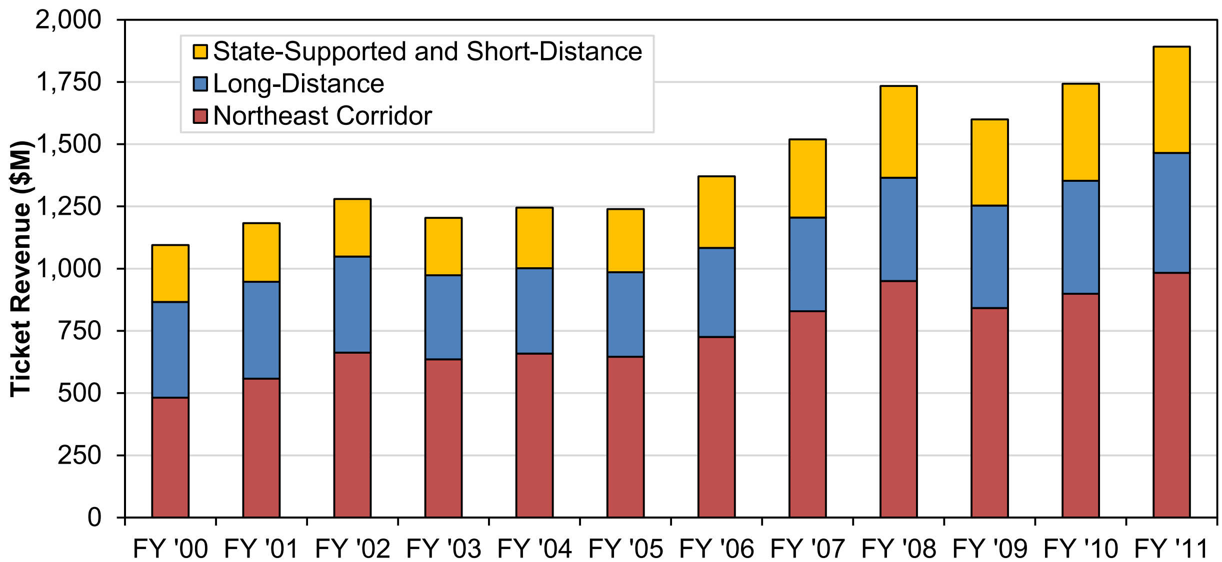


Figure 1.2: Amtrak Revenue Since Year 2000 (Amtrak, 2012)

These figures lend support to the argument that intercity passenger rail is increasingly relevant in a transportation market that has for decades been dominated by highways and commercial aviation. Also apparent is the great potential for new state supported and short-distance services. These services, often characterized with conventional speeds and low frequency of operation have, in total, eclipsed the ridership of the high speed, high frequency Northeast Corridor services. With better frequency and speed on short distance and state supported corridors, it stands to reason that these services would experience further ridership and revenue gains. The problem faced by intercity rail planners is how to increase the service level of these corridors in the most cost effective manner possible.

Figures 1.3 and 1.4 show the national route network of intercity passenger rail service in 1962 and 2005 respectively. In these figures, red lines indicate at least six daily round trips on a particular corridor and bold black lines indicate at least three daily round trips. Although the network coverage of 1962 is far more extensive than present day, it can be noted that several corridors in the Northeast, Midwest, and Western regions retained high service frequencies through the transition from class 1 railroad to Amtrak operated passenger services. These surviving corridors generally connect cities with high population density and congested competing transportation infrastructure that causes rail service to be more attractive to travelers.

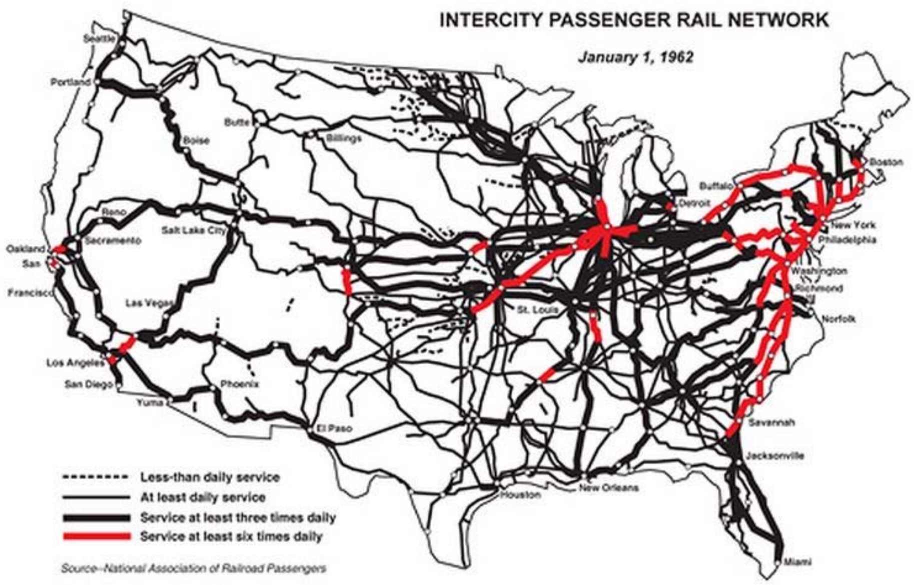


Figure 1.3: Intercity Rail Service in Year 1962 (Alpert and Kenton 2011)

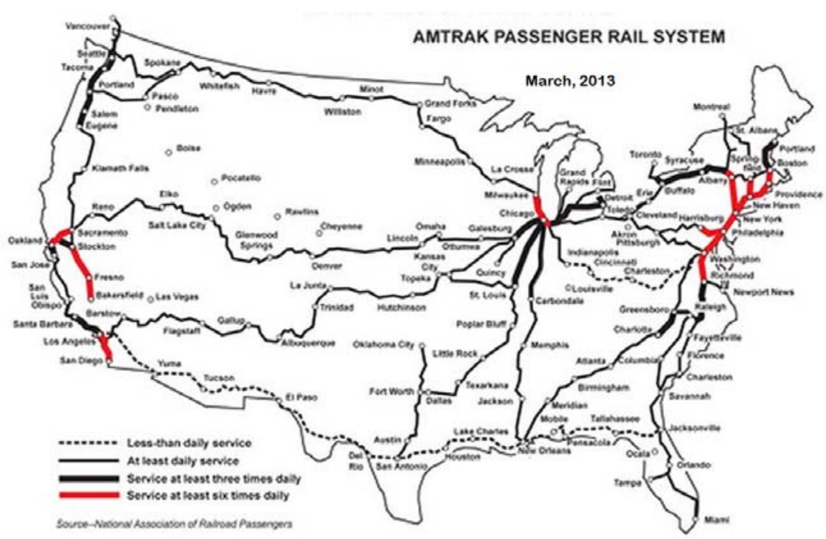


Figure 1.4: Amtrak Service Network Year 2005 (Alpert and Kenton 2011)

Table 1.1 is a comparison of various short-distance and state-supported corridors. Ridership on any given route may be influenced by numerous interacting factors, including the demand for passenger travel as well as the costs and levels of service of various transport modes. This table serves to illustrate corridor-to-corridor variations of level of service and ridership. If ridership is judged as a barometer of the success of a corridor, then it can be observed from the table that corridors with higher frequency, reliability, and greater average speed are generally more successful. The Midwestern and Western routes shown in the table are sorted by average end-to-end speed, a figure that includes any buffer time built into the schedule.

Table 1.1: Comparison of Amtrak Regional Routes

Service Name	Total Dist. (mi)	Time (hr:mm)	Max. Spd. (MPH)	Avg. Spd. (MPH)	Int. Stops	Avg. Stop Spacing (mi)	Trains per Day	OTP.[1]	Yearly Ridership[2]
Midwestern Routes									
Hiawatha	86	1:29	79	58.0	3	21.5	14	96.7%	838,355
Illinois Zephyr	258	4:28	79	57.8	8	28.7	4	92.7%	232,592
Illini, Saluki	309	5:30	79	56.2	9	30.9	4	70.8%	325,255
Lincoln Service	284	5:25	110	52.4	9	28.4	8	92.3%	597,519
Wolverine	304	6:02	110	50.4	15	19.0	6	55.0%	484,138
Blue Water	319	6:38	110	48.1	9	31.9	2	62.9%	189,193
Pere Marquette	176	4:00	79	44.0	3	44.0	2	58.1%	109,321
Hoosier State	196	5:05	79	38.6	4	39.2	2	82.0%	36,669
Western Routes									
San Joaquin	315	5:22	79	58.7	13	22.5	8-12	80.6%	1,144,616
Capital Corridor	168	3:08	79	53.6	13	12.0	15-30	94.4%	1,746,397
Pacific Surfliner	350	8:20	90	42.0	27	12.5	20	78.9%	2,640,342
Cascades	457	11:10	79	40.9	16	26.9	4-8	65.3%	845,099

[1] 12 month period from February 2013, February 2014 for Western Routes
[2] Based on Amtrak Fiscal Year 2012

1.2 Freight Rail Perspective on Passenger Rail

One of the unique aspects of American intercity passenger rail is the institutional framework in which passenger trains operate. Unlike many developed countries with government owned rail networks, much of the trackage passenger trains operate over in the U.S. is owned and maintained by private companies. These companies, which in most cases are class 1 railroads, are primarily focused on the safe and profitable transportation of freight, and their networks are optimized accordingly. Passenger trains consume capacity and create operational challenges that interfere with the primary business of moving freight. Class 1 freight railroads carefully consider their current and potential future freight railroad franchise when evaluating a proposed passenger service. To preserve this franchise, the agency sponsoring the new service must make infrastructure investments to mitigate any capacity impacts (Rose 2008, UPRR 2012).

Although the methodology outlined later in this work could be used to help plan passenger corridors, a particular infrastructure configuration can only be suggested, and not dictated, in the planning of improved passenger service. Class 1 railroads have ultimate authority over capacity and performance improvements to their networks. The burden of proof lies with the passenger agency to demonstrate that any service changes will preserve safety, limit freight carrier liability, and not harm the freight transportation franchise. For many corridors this precludes passenger trains operating at speeds greater than 90 MPH on trackage shared with freight trains. With this constraint, it is all the more important to consider a holistic scope of improvements and not only those that improve running time by increasing route maximum passenger service speed.

1.3 Discussion of Running Time Components

The total commercial or public schedule that is presented to the rail passenger can be developed with three principal time components. The first and largest of these is the minimum running time of a specific train consist between two points. This time assumes ideal weather, driver performance, rolling stock mechanical condition, and no interference from other rail traffic. Minimum running time can be improved through changes in the maximum speed of track segments, or through the use of rolling stock with better curving, acceleration, or braking performance.

Any time in addition to this minimum running time is considered delay. It is important to distinguish delay from the perspective of a passenger versus delay from the perspective of the minimum run time. A train might experience delay and still, from the perspective of the passenger, arrive on time or even early due to slack time built into the commercial schedule. For the purposes of this thesis, delay is considered relative to the minimum running time and not the commercial schedule.

The second schedule component is an expected amount of planned delay that is added at different locations. For example, an expected dwell of several minutes might be added at certain stations to allow for the boarding and alighting of passengers. In addition, a meet with an opposing passenger train on single-track territory might add further dwell time to a schedule between two stations.

Passenger schedules may also have a third time component - slack time for unplanned delays. This is intended to mitigate the impacts of mechanical failures, signal failures, slow orders, unanticipated conflicts with other rail traffic, or higher than expected passenger delays. For these reasons, many Amtrak schedules have an amount of buffer time equal to 8% of the minimum running time added to the last station on the route (Franke et al. 2008).

The total slack time added to a service schedule should be carefully considered in the planning phases of a passenger rail project. Although this thesis primarily focuses on improvements that impact the minimum running time of a schedule, improvements that impact slack time may in some cases be a more cost effective way of improving the overall commercial schedule.

1.4 Planning and Engineering of Intercity Rail Corridors

Intercity rail corridors are generally planned by government agencies in partnership with Amtrak. Before designing or permitting infrastructure improvements for an intercity corridor, the passenger organization will typically commission a route feasibility study to determine ridership, revenue, and the costs of any improvements necessary to implement changes to the service (Franke and Hoffman 2007, Franke et al. 2008, and Amtrak 2009b). Amtrak will collaborate with host railroads to determine the scope of any infrastructure improvements. In this process, a package of capacity and speed upgrades will be agreed upon as a condition for allowing new passenger service. Figure 1.5 shows the summary of these proposed improvements for new passenger service from Chicago, IL to Iowa City, IA.

II.A.1.iii. Summary of Capital Costs - Chicago–Quad Cities–Iowa City Route

Via Route A:

	$ Millions		
	A4 As-is	A5 60 mph	A6 79 mph
Total Capital Cost – Iowa Segment Only (1)	$0.3	$26.1	$32.5
Total Capital Cost – Illinois Segment Only (2)	5.6	13.8	22.4
Grand Total - Route A	$5.9	$39.9	$54.9

Via Route B:

	$ Millions		
	B4 As-is	B5 60 mph	B6 79 mph
Total Capital Cost – Iowa Segment Only (1)	$0.3	$ 26.1	$ 32.5
Total Capital Cost – Illinois Segment Only (2)	-	78.4	93.8
Grand Total - Route B	$0.3	$104.5	$126.3

Footnotes:

(1) Iowa segment includes $0.3 million for Iowa City layover facility
(2) Illinois segment excludes projected funding for a layover facility

Figure 1.5 – Typical Feasibility Study Cost vs. Speed Comparison (Franke et al. 2008)

The study authors used their industry experience to determine which segments were best suited for upgrading to higher speeds. For feasibility studies, the scope of improvements is considered at a high level. Decisions about specific projects, such as realigning a curve or incrementally improving the speed of several small segments over time are typically left to consultants after the corridor improvement scope is defined, funded, and engineering design is underway.

With marginal additional effort, passenger agencies could have earlier access to more detailed information about the relationship between cost and corridor performance. This would enable agency planners to more effectively allocate improvement capital between different routes, thereby maximizing service benefits for the investment made. Without this strategic view of rail corridors, there is no clear understanding of the relationship between improvement cost and benefit leading to the potential for inefficient investments and a waste of public resources.

This author believes that agencies should develop a more strategic view to corridor upgrades and embrace modest, year over year improvements as part of a long range plan. Some states, such as Washington, have already embraced this methodology (ODOT 2013). Numerous other states that support short distance passenger service lack a publically defined strategy with regards to service improvement.

1.5 Conclusions

Contemporary intercity passenger rail in the United States includes long distance as well as regional services. Although the contemporary network of passenger corridors is less extensive than in the 1960s, key corridors of high train frequencies have weathered political, economic, and operating challenges to the present time. If intercity passenger rail is to continue to make competitive gains in the transportation market, policy makers must expect to further improve the level of service through increased passenger train frequency, reliability, and speed. In many cases, these improvements must be made while conforming to the safety, capacity, and legal requirements of host freight railroads. Finally, passenger rail agencies must drive the design and execution of service improvements within the context of a long-term strategic plan.

CHAPTER 2: A SPREADSHEET BASED TRAIN PERFORMANCE CALCULATOR

2.1 Introduction

A train performance calculator (TPC) is a software tool used to determine the running time of a train over a rail route. A TPC typically requires input parameters related to route and train physical characteristics. Using this information, the software will compute a best case speed versus distance performance. Train performance calculators are a mature technology with several programs commercially available and validated by real-world data. Despite this, commercial TPCs are costly and may be difficult to modify to reflect specific route or rolling stock combinations. These difficulties motivated researchers at the University of Illinois at Urbana-Champaign to pursue development of an easy-to-modify TPC based in a commonly available, low cost spreadsheet software.

The first version of this program was developed in late 2011 to compute running time calculations for a high-speed rail operations analysis. The initial spreadsheet considered pre-determined acceleration and braking times between different speeds and did not directly base its calculations on the physics of train acceleration and braking. As a consequence, it did not consider the impacts of grade and curvature. The program was subsequently refined to enable more detailed calculations of running time, including these two parameters. This refined second generation TPC spreadsheet is what is described here.

The TPC is coded in Microsoft Excel as a Visual Basic for Applications (VBA) script and can compute train running time over several hundred miles of route generally in less than 5-10 minutes of processing time. The spreadsheet features four separate scripts and user interface tabs. Three additional tabs provide dedicated space for the spreadsheet to perform acceleration and braking calculations. The main interface tabs include a route tab, rail vehicle tab, a calculator tab, and a speed vs. distance plot tab. The route and rail vehicle tabs are used to load the appropriate train and route parameters into the spreadsheet. The calculator tab illustrates the TPC process and displays the calculation results in a tabular format. The speed vs. distance tab collects results from the calculation sheet and displays a plot illustrating the speed of the train along the route.

Using the spreadsheet described in this work, planners of passenger rail corridors can perform relative assessments of different infrastructure, rolling stock, and train operating scenarios and expect rapid and reasonably accurate results with the convenience and economy of a ubiquitous spreadsheet software. This work describes the functions of the TPC and presents a case study application of the spreadsheet to analyze various improvement scenarios for an intercity passenger rail corridor.

2.2 Literature Review

This section describes prior work of individuals and research groups that is relevant to train resistance and the use of train performance calculators.

Davis (1926) considered and synthesized previous work on train resistance conducted by General Electric, the Pennsylvania Railroad, the Chicago Burlington and Quincy Railroad, and Professor Edward Schmidt of the University of Illinois. Using this previous research, Davis developed a unified train resistance formula, commonly referred to as the Davis equation. This formula has been the foundation of nearly all subsequent studies into train resistance and is a necessary component for calculating the kinematic behavior of a train operating along a route.

Smith (1951) described an early type of train performance calculator used by the Pennsylvania Railroad. The machine used electrically interconnected curve drawing instruments to plot train speed and distance travelled. This machine was used to analyze the performance of different types of motive power assigned to various freight and passenger train services. The author concluded that the TPC has been successful in saving time over previous manual calculation techniques and has allowed opportunities for investigating train performance questions that would otherwise not have been practical.

Avery (1985) demonstrated various graphical representations of data produced by TPC software. The author used graphical results of TPC outputs to illustrate the impact of speed restrictions, power, grades, and curves on train performance. The author concluded that graphical representations of TPC outputs such as performance and energy consumption can be useful in optimal design of transit, intercity, and freight railway corridors.

Krueger et al. (2000) reviewed the uses of simulation railway operations analysis and discussed various problems associated with their application. A TPC is described as an integral first step in any railway simulation process. The role of the TPC in a broader simulation framework is to first establish the maximum speed of a train service given information on grades, curves, speed restrictions, power, and train resistance. Further layers of the railway simulation process must use the TPC stage in order to determine the impacts of other operating factors such as signaling, dispatching, and the management of train operating crews.

Lukaszewicz (2006) conducted full scale running resistance tests to determine the relationship between train resistance and speed for different train consists. Data were collected in Sweden with a high speed X2 trainset, a conventional passenger train, and a freight train. The author found that train running resistance can be expressed in general form by a second order polynomial. He found that the coefficients of the polynomial are dependent on characteristics of rolling stock and track structure. The A coefficient of the first term is dependent on axle load, number of axles, and the type of track. The B coefficient varies with train length and train speed. The C coefficient is related to aerodynamic drag and depends on the length of train and the configuration of the front and rear vehicles. The results of this work are generally consistent with the formulas developed by Davis in 1926, although tailored specifically to the examples of Swedish rolling stock studied.

Wardrop (2009) described the uses of various analytical tools related to planning, engineering, and operational aspects of railways in Australia. The author discusses various uses of TPC software in analyzing train configuration and performance as well as infrastructure configurations and line capacity. The author comments that many TPC programs have been developed to model a specific type of railway operation and any one model might not be easily adopted as a general evaluation tool. In addition, the author cautions that TPC software should be regularly calibrated against recorded train performance data in order to ensure accuracy.

2.3 A Spreadsheet Based Train Performance Calculator

The following section describes the functions of each component of the TPC spreadsheet along with an overview of calculations performed. Screen captures are provided throughout this section in order to help orient the reader with the spreadsheet user interface. The series of calculations needed in order for the TPC to function are divided for reasons of clarity onto several different spreadsheet tabs. The tab calculations should be performed in the following order: 1) route calculation, 2) rail vehicle calculation, 3) train performance, and finally 4) speed vs. distance plot generation.

Route Tab – The route tab interface is illustrated in Figure 2.1. On this tab railway line data are entered into the spreadsheet. The route is separated into discrete segments defined by changes in speed, curvature, gradient, or the presence of stations stops along the route. This information can typically be gathered from railroad operating timetables, track charts, or GIS databases of rail infrastructure. Approximate simulations can also be performed with conservative engineering judgment and approximate segment lengths measured from satellite image and mapping software such as Google Earth Pro. Station stops are indicated by including a route segment of zero length with a maximum speed of zero. Dwell time can also be added to the spreadsheet on the same row as the station stop. The cumulative milepost and equivalent grade columns are computed by activating the route calculation. The equivalent grade is a method of including the effects of curvature in train resistance calculations. In this method, curve resistance is converted into an equivalent amount of grade resistance (Hay 1982). The equivalent grade column adds 0.04% of grade for each degree of curvature on the segment. For example, a 10 degree curve on tangent track would result in the same train resistance as a 0.4% grade taken in isolation. After running the script on this tab that performs the necessary route calculations, the spreadsheet user may proceed to the rail vehicle tab. The route VBA script is detailed in Appendix C.

Cat.	Name	Length (miles)	Cumul. MP (miles)	Speed (MPH)	Grade (%)	Curve (deg)	Dwell (min)	Equiv. Grade (%)
		0.00	0.00	0	0.00	0.00		0.00
		1.00	0.00	60	0.00	0.00		0.00
		1.00	1.00	20	-0.83	0.00		-0.83
		0.70	2.00	20	-0.83	1.00		-0.79
		0.30	2.70	20	-0.83	1.00		-0.79
		0.30	3.00	60	-0.83	0.00		-0.83
		0.45	3.30	60	-0.83	3.00		-0.71
		0.15	3.75	60	-0.83	0.00		-0.83
		0.40	3.90	60	-0.83	0.00		-0.83
		0.70	4.30	60	-0.50	0.00		-0.50
		0.10	5.00	60	-0.50	2.00		-0.42
		0.40	5.10	60	-0.50	0.00		-0.50
		1.00	5.50	60	-0.50	1.00		-0.46
		0.80	6.50	60	0.02	0.00		0.02
		0.38	7.30	60	0.02	0.00		0.02
		0.07	7.68	20	0.02	4.33		0.19
		0.30	7.75	20	0.02	0.00		0.02
		0.10	8.05	20	0.02	2.00		0.10
		0.08	8.15	20	0.02	0.00		0.02

Figure 2.1: TPC Route User Interface

Rail Vehicle Tab – The rail vehicle tab is illustrated in Figure 2.2. On this tab, the performance data for the consist of rail vehicles are entered into the spreadsheet. The script executed on this sheet computes the total train resistance parameters considering the individual characteristics of the different vehicle components.

Name	Weight (tons)	Length (ft)	Traction Coef.	Power (hp)	Davis A Coef.	Davis Aa Coef.	Davis B Coef.	Cross Section Area (ft^2)	Davis CN Coef.	Number of Axles	Max Speed (MPH)	Sub. Total A (lbs)	Sub. Total B (lbs/mph)	Sub. Total C (lbs/mph^2)	Starting TE (lbs)
P42DC	134.12	69	0	-	1.5	18	0.03	110	10	4	110	273	4.02	0.11	-
P42DC	134.12	69	0.3	4,250	1.5	18	0.03	110	3.5	4	110	273	4.02	0.04	80,472
Amfleet I	58	85	0	-	1.5	18	0.03	110	3.5	4	125	159	1.74	0.04	-
Horizon car	57	85	0	-	1.5	18	0.03	110	3.5	4	125	158	1.71	0.04	-
Horizon car	57	85	0	-	1.5	18	0.03	110	3.5	4	125	158	1.71	0.04	-
Horizon car	57	85	0	-	1.5	18	0.03	110	3.5	4	125	158	1.71	0.04	-
Horizon car	57	85	0	-	1.5	18	0.03	110	3.5	4	125	158	1.71	0.04	-
Amfleet I	58	85	0	-	1.5	18	0.03	110	3.5	4	125	159	1.74	0.04	-

Brennan.Caughron
Click to calculate tota

Total Characteristics	
Train length (feet)	648
Train length (miles)	0.12
Max. speed (MPH)	110
Train weight (tons)	612
Train weight (lbs)	1,224,480
Train mass (slug)	38,058
Train start TE (lbs)	80,472
Train power (hp)	4,250
Train A (lbs)	1,494
Train B (lbs/mph)	18.37
Train C (lbs/mph^2)	0.38
Route grade interval (miles)	0.05
Train grade interval (fraction)	10
Max. permissible accel. (mph/s)	3.00
Max. permissible decel. (mph/s	3.00
HP to TE efficiency (%)	0.90

Figure 2.2: TPC Rail Vehicle User Interface

Each vehicle in the train consist is added to the sheet as a separate row. For cases of electric multiple unit or diesel multiple unit trains, the characteristics of the entire train can be loaded into the spreadsheet as one row. The train resistance parameters are based on the Canadian National train resistance formula (Equation 1) (AREMA 2010a). This formula is a modification of the 1926 Davis equation.

$$R_r = 1.5 + \frac{18N}{W} + 0.3V + \frac{CaV^2}{10000W} \tag{1}$$

In this equation, R_r is the unit rolling resistance in lbs/ton. N is the number of axles for the vehicle being considered. W is the total weight in tons of a locomotive or car. The parameter V is the velocity of the train in MPH. C is the streamlining coefficient specific to the CN formula. The parameter a is the cross sectional area of the locomotive or car in square feet.

$$TF = \frac{375Pe}{V} \tag{2}$$

Tractive force was computed for each powered vehicle using Equation 2 (Hay 1982). In this equation, the tractive force TF in lbs is computed by multiplying 375 by the horsepower P, an efficiency factor e, and dividing by the velocity V in MPH. The efficiency factor was assumed as 0.90 for the examples presented in this work, but can be altered for different vehicles or scenarios. At low speeds, the tractive force generated by a locomotive is limited by the adhesion at the wheel rail interface (Hay 1982). This low speed or starting tractive force is a function of the weight on powered axles and is determined in this TPC spreadsheet by a traction coefficient multiplied by the weight of the powered vehicle. The total characteristics of all rail vehicles in the consist are computed by activating the script on this sheet. The detailed rail vehicle script is shown in Appendix C. After completing the rail vehicle script, the user may proceed to the train performance calculator tab.

Train Performance Calculator Tab – The TPC tab (Figure 2.4), uses the parameters generated by the route and rail vehicle tabs to compute the best case running time of a train over the route. The detailed TPC script is shown in Appendix C. When this script is activated, each route segment and its associated physical characteristics are copied from the route tab to the calculator tab. At any given location along a route, a train may span several different track segments of positive and negative sloped gradients. After copying the route parameters the TPC computes the average gradient of the train as a function of train position along the route (Figure 2.3). This function is referenced later in the calculation process each time the TPC simulates train acceleration or braking between route segments of different operating speeds. The distance interval and train segment interval used for this calculation can be specified in the rail vehicle sheet. Smaller intervals increase the accuracy of the average grade function but lead to longer processing times for given computing capacity.

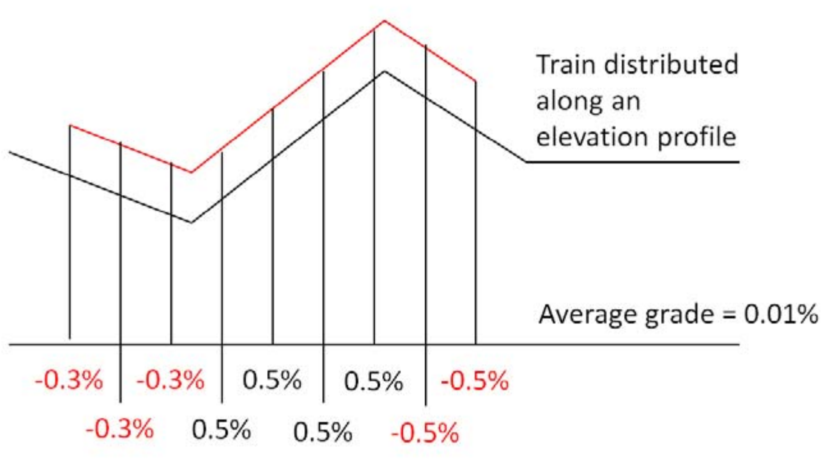


Figure 2.3: Illustration of Train Average Gradient

The examples presented in this work use a route distance interval of 0.05 miles and a train interval of 10, meaning the train length is divided into 10 segments for the average grade calculation. The average grade information is saved on the tab titled "Sub2" for later reference by the script in acceleration and braking calculations. A key simplifying assumption of the present version of the TPC spreadsheet is that the train consist has a constant linear weight density. Consideration should be given to improving this aspect in future revisions of the model.

After computing the average grade for the train consist as it traverses the route, the TPC script then starts by computing the acceleration and braking distances and times for each change in speed between route segments. This script is activated on the TPC interface shown in Figure 2.4.

Cat.	Name	Length (miles)	Cumul. MP (miles)	Speed (MPH)	Grade (%)	Curve (deg)	Dwell (min)	Equiv. Grade	Adjusted Speed (MPH)	Acel. from Prev. (miles)	Decel. to Next (miles)	Cruise Distance (miles)	Accel. Time (min)	Decel. Time (min)	Cruise Time (min)	Total Time (min)	Cumul. Time (min)
	Chicago Union	0	0	0	-	-	0	-	0	-	-	-	-	-	-	-	-
	U. Station to C	0.57	0	15	-	-		-	15	0.03	-	0.54	0.2	-	2.2	2.4	-
	CP Canal to Cli	0.07	0.57	15	-	-		-	15	-	-	0.07	-	-	0.3	0.3	2.4
	Clinton to Jeff	0.05	0.64	30	-	-		-	25	0.05	-	0.00	0.2	-	0.0	0.2	2.7
	Jefferson to G	0.34	0.69	30	-	-		-	30	0.05	-	0.29	0.1	-	0.6	0.7	2.9
	Green to CP M	0.22	1.03	30	-	-		-	30	-	-	0.22	-	-	0.4	0.4	3.6
	CP Morgan to	0.2	1.25	30	-	-		-	30	-	-	0.20	-	-	0.4	0.4	4.0
	Racine to Paul	0.68	1.45	60	-	-		-	57	0.64	-	0.04	0.9	-	0.0	0.9	4.4
	Paulina to Lea	0.53	2.13	60	-	-		-	60	0.15	0.09	0.29	0.2	0.1	0.3	0.6	5.3
	Leavitt to Tow	0.31	2.66	60	-	-		-	54	-	0.30	0.01	-	0.5	0.0	0.5	5.9
	Tower A-2	0.1	2.97	20	-	-		-	20	-	-	0.10	-	-	0.3	0.3	6.4
	Tower A-2 to I	2.3	3.07	60	-	-		-	60	0.84	-	1.46	1.2	-	1.5	2.6	6.7
	MP 5.4 to Gray	2.7	5.37	79	-	-		-	79	1.30	0.33	1.07	1.1	0.3	0.8	2.2	9.3
	Grayland	0.1	8.07	60	-	-		-	60	-	-	0.10	-	-	0.1	0.1	11.6
	Grayland to M	0.7	8.17	79	-	-		-	69	0.50	0.14	0.06	0.5	0.2	0.0	0.7	11.7
	Mayfair	0.1	8.87	60	-	-		-	60	-	-	0.10	-	-	0.1	0.1	12.4
	Mayfair to Gle	8.43	8.97	110	-	-		-	109	6.73	1.39	0.30	4.5	1.6	0.2	6.3	12.5
	Glenview	0	17.4	0	-	-		-	0	-	-	-	-	-	-	-	18.7
	Glenview to La	10.9	17.4	110	-	-		-	110	7.95	1.42	1.52	6.1	1.6	0.8	8.6	18.7
	Lake Forest	0	28.3	0	-	-		-	0	-	-	-	-	-	-	-	27.3
	Lake Forest to	3.97	28.3	110	-	-		-	93	3.92	-	0.05	3.8	-	0.0	3.8	27.3
	MP 32.3 (Rond	0.1	32.27	110	-	-		-	93	-	-	0.10	-	-	0.1	0.1	31.1
	MP 32.3 to Stu	30.7	32.37	110	-	-		-	110	4.04	1.42	25.24	2.4	1.6	13.8	17.8	31.2
	Sturtevant	0	63.07	0	-	-		-	0	-	-	-	-	-	-	-	48.9
	Sturtevant to I	15.4	63.07	110	-	-		-	110	7.95	-	7.45	6.1	-	4.1	10.2	48.9
	Milwaukee Ai	0	78.47	110	-	-		-	110	-	-	-	-	-	-	-	59.1
	M. Airport to I	1.9	78.47	110	-	-		-	110	-	0.85	1.05	-	0.6	0.6	1.2	59.1
	MP 80.3 - MP 8	3	80.37	70	-	-		-	70	-	0.39	2.61	-	0.5	2.2	2.7	60.3
	MP 83.3 - MP 8	1.2	83.37	40	-	-		-	40	-	0.09	1.11	-	0.2	1.7	1.8	63.0
	MP 84.5 - MP 8	0.4	84.57	30	-	-		-	30	-	0.09	0.31	-	0.3	0.6	0.9	64.8
	MP 84.9 - MP 8	0.6	84.97	15	-	-		-	15	-	0.03	0.57	-	0.3	2.3	2.5	65.7
	Milwaukee	0	85.57	0	-	-	0	-	0	-	-	-	-	-	-	-	68.2

Figure 2.4: TPC Main Calculation Sheet

Acceleration and braking distances and times are computed by a series of numerical integrations. Speed at each time interval is computed by integrating train acceleration or braking. Further integrating the speed at each time step determines the distance covered by the train in each interval. The spreadsheet uses one-second time intervals for these calculations. These numerical acceleration and braking integrations are performed on a sub-sheet named “Sub.” The use of this sub sheet shows numerical figures of the integration at different stages of the calculation, which can be useful for the user to pause the calculation script and check the TPC for any errors. At the end of each calculation, acceleration or braking distances and times are copied to the calculator sheet before the sheet “Sub” is cleared for the next calculation. The sheet named “Sub3” collects a history of all time step calculations performed in the current TPC scenario and is used in creating the final speed vs. distance plot.

Once the TPC has completed acceleration and braking calculations for all route segments, the spreadsheet then determines whether or not the speed profile presented by the maximum speed on each segment is feasible for the given train consist. For each segment, the distance that the train is travelling at maximum speed is computed. This “cruising distance” is equal to the segment length, minus the distance necessary to accelerate from the previous segment, minus the distance necessary to brake to the next segment. For segments where there is no change in speed, both the acceleration distance from the previous segment, and the braking distance to the next segment, are equal to zero. When all of the segment cruise distances are greater than zero, the speed profile is feasible. When one or more of the segments features a negative cruise distance, the train cannot match the speed profile proposed by the maximum target segment speeds. When this occurs, the TPC script will iteratively decrease the maximum target speed in one MPH increments on segments where the cruise distance is negative and re-calculate the acceleration and braking distances for the current and adjacent segments. These iterative changes to the train target speed are recorded in the "adjusted speed" column. The script will iterate through the route segments as many times as necessary to produce a positive cruise distance on all segments.

After a feasible speed profile is determined, the TPC script will calculate the total running time for each segment. This total time is the sum of the time necessary to accelerate from the previous segment speed, cruise at maximum speed, and brake to the next segment speed. A cumulative time column indicates when a train enters each segment and is calculated by summing all of the previous total segment times. For station stop segments, the amount of dwell time included in the route tab is added to both the total and cumulative time columns for the appropriate segments. The script is complete once the cumulative times have been calculated for the whole route. In the approximately 50-segment-long routes evaluated in this research, computation times may range from 5-10 minutes for one scenario on a quad-core 3.4 GHz processor desktop computer with 16GB for RAM. Segments with more routes would be expected to take longer to process. Once the TPC script has completed, the user may proceed to the speed vs. distance sheet.

Speed vs. Distance Plot Tab – The principal outputs of the entire TPC spreadsheet are the tabular results generated in the calculator tab and the graphical display of the speed vs. distance performance of the train generated in the tab named "SpeedDist." The speed vs. distance script will copy the appropriate data from the "Sub3" acceleration and braking calculation record sheet. These data are filtered and sorted at the end of the TPC calculator script in order to include only the acceleration and braking calculations that are reflected in the final feasible speed profile. The VBA code for this calculation is shown in Appendix C.

2.4 Case Study Scenario

A case study scenario is presented to illustrate the functionality of the TPC spreadsheet in evaluating different infrastructure improvement scenarios for an intercity passenger railway service. The route selected for this example is the Amtrak *Hiawatha* service that operates seven round trips per day over 86 miles of railway between Chicago, IL and Milwaukee, WI. The existing rail service operates using diesel-electric locomotive powered trains of six single-level passenger coaches. The current maximum speed of the service is 79 MPH, and the minimum scheduled time from end to end is 89 minutes including three intermediate stops. Subtracting buffer time at each end of the schedule, the minimum running time including intermediate stops is 84 minutes. There have been recent proposals to add a station stop in Lake Forest, IL between the present stops of Glenview, IL and Sturtevant, WI (Nelson 2012). The following analysis assumes that this stop has been added to the train service.

Several improvement scenarios were created for the present corridor. Table 2.1 summarizes these scenarios along with the present route conditions. The improvement scenarios consider two separate route segments and two upgraded higher speed scenarios. Scenario D1 reflects the baseline condition of the corridor with conventional 79-MPH maximum operating speeds. Scenarios D2 and D3 consider the upgrade of two route segments to 110-MPH maximum speed with passenger services operated by the same diesel locomotive hauled trainsets as in the baseline scenario. The total consist tractive force and train resistance curves used in later TPC calculations are shown in Figure 2.5. The rail vehicle characteristics used to construct Figure 2.5 are shown in Appendix Table C.2.

Table 2.1: Comparison of Route Characteristics

Segment Description	Seg. Length (mi.)	Milepost	Segment Speed (MPH)				
			Diesel Loco. Consist			E. Mult. Unit	
			D1	D2	D3	E1	E2
Chicago Union Station (s)	0.00	0	0	0	0	0	0
U. Station to CP Canal	0.57	0.57	15	15	15	15	15
CP Canal to Clinton	0.07	0.64	15	15	15	15	15
Clinton to Jefferson	0.05	0.69	30	30	30	30	30
Jefferson to Green	0.34	1.03	30	30	30	30	30
Green to CP Morgan	0.22	1.25	30	30	30	30	30
CP Morgan to Racine	0.20	1.45	30	30	30	30	30
Racine to Paulina	0.68	2.13	60	60	60	60	60
Paulina to Leavitt	0.53	2.66	60	60	60	60	60
Leavitt to Tower A2	0.31	2.97	60	60	60	60	60
Tower A-2	0.10	3.07	20	20	20	20	20
Tower A-2 to MP 5.4	2.30	5.37	60	60	60	60	60
MP 5.4 to Grayland	2.70	8.07	79	79	79	79	79
Grayland	0.10	8.17	60	60	60	60	60
Grayland to Mayfair	0.70	8.87	79	79	79	79	79
Mayfair	0.10	8.97	60	60	60	60	60
Mayfair to Glenview	8.43	17.4	79	79	110	79	110
Glenview (s)	0.00	17.4	79	79	110	79	110
Glenview to Lake Forest	10.90	28.3	79	79	110	79	110
Lake Forest (s)	0.00	28.3	79	79	110	79	110
Lake Forest to MP 32.3	3.97	32.27	79	110	110	220	220
MP 32.3 (Rondout)	0.10	32.37	79	110	110	220	220
MP 32.3 to Sturtevant	30.70	63.07	79	110	110	220	220
Sturtevant (s)	0.00	63.07	79	110	110	220	220
Sturtevant to M. Airport	15.40	78.47	79	110	110	220	220
Milwaukee Airport (s)	0.00	78.47	79	110	110	220	220
M. Airport to MP 80.3	1.90	80.37	79	110	110	220	220
MP 80.3 - MP 83.3	3.00	83.37	70	70	70	70	70
MP 83.3 - MP 84.5	1.20	84.57	40	40	40	40	40
MP 84.5 - MP 84.9	0.40	84.97	30	30	30	30	30
MP 84.9 - MP 85.5	0.60	85.57	15	15	15	15	15
Milwaukee (s)	0.00	85.57	0	0	0	0	0

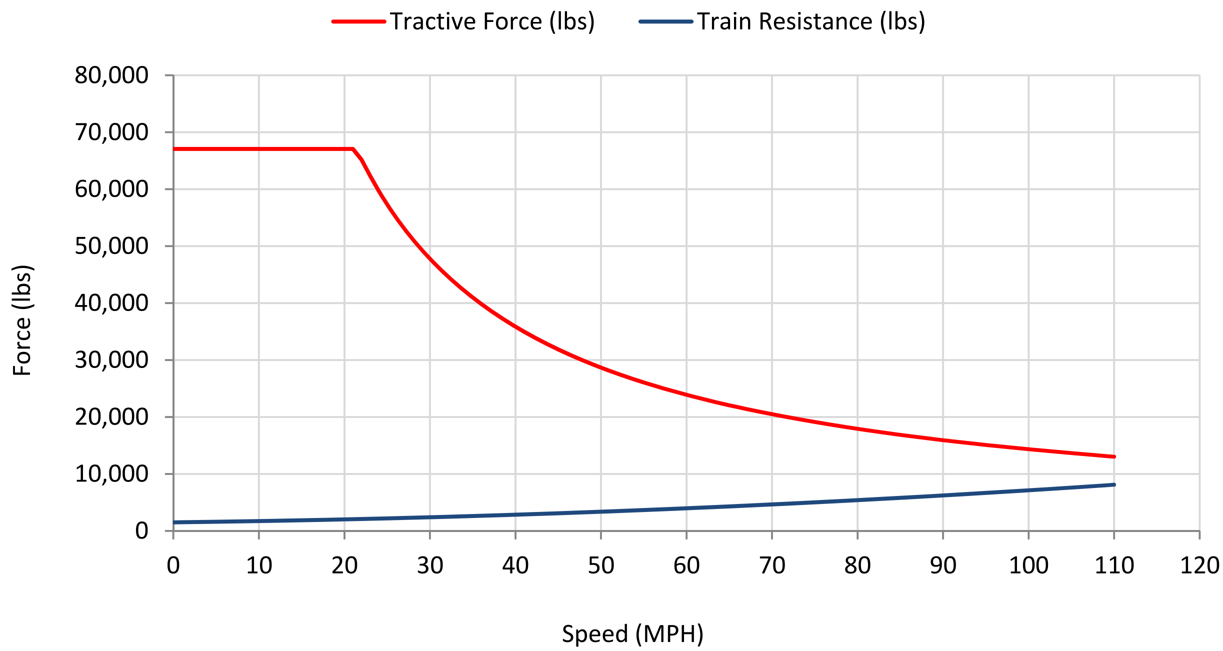


Figure 2.5: Diesel Locomotive Tractive Effort vs. Train Resistance Curves

Scenarios E1 and E2 consider the construction of a new, dedicated 220-MPH high-speed electrified rail line along a segment of the route where the present right-of-way is unlikely to be constrained by adjacent land development. These scenarios would involve high-speed trainsets operating at slower speeds while sharing trackage with commuter and freight services and operating at very high speeds on the central dedicated segment. Scenario E2 also includes an upgraded segment adjacent to the 220 MPH segment where the rail service would operate at speeds up to 110 MPH while sharing trackage with freight and other passenger traffic. Both E1 and E2 scenarios would use an EMU trainset for intercity passenger service. The tractive force and train resistance curves used in the EMU TPC calculations are taken from the specifications of the Valero-E trainset manufactured by Siemens (2013) (Figure 2.6). The rail vehicle characteristics used to construct equivalent tractive force and train resistance functions for use in the TPC spreadsheet are shown in Appendix Table C.3.

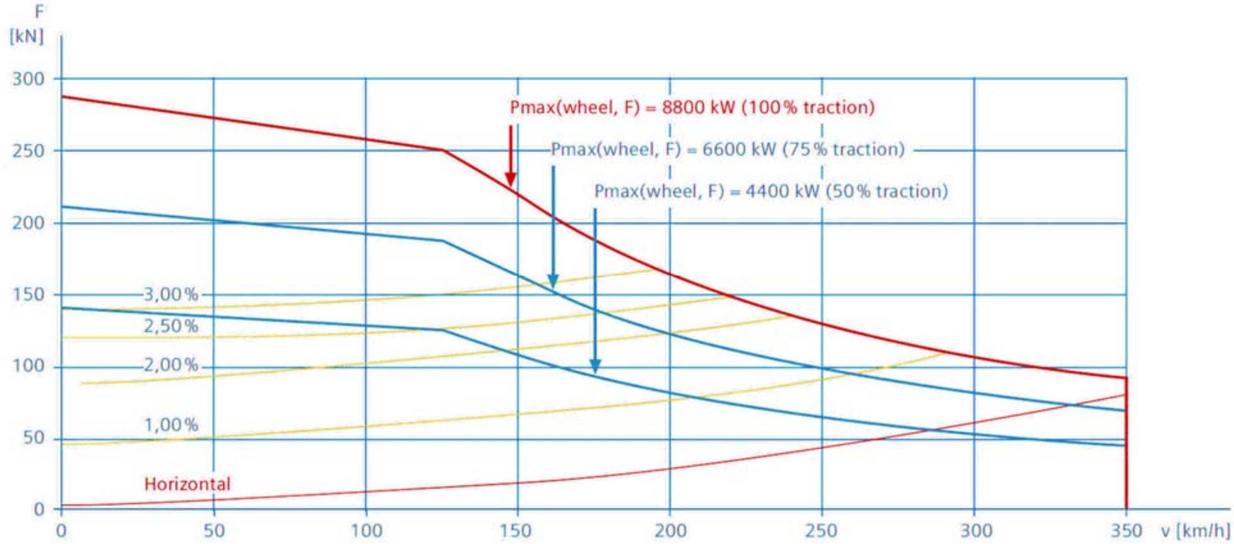


Figure 2.6: EMU Tractive Effort vs. Resistance Curves (Siemens 2013)

All improvement scenarios considered express service that would operate from terminal to terminal without stopping at any intermediate stations. A local service was also analyzed with stops at all intermediate stations on the existing schedule. A dwell time of one minute was used for each intermediate station stop to allow for passenger boarding and alighting. All scenarios were loaded into the TPC spreadsheet and minimum running times were computed for the northbound trip from Chicago to Milwaukee. The resulting train speeds vs. distance plots generated by the spreadsheet are shown in Appendix Figures C.1 through C.10. The summary running times for each of the scenarios are shown in Table 2.2. The scenarios are sorted from top to bottom by decreasing terminal-to-terminal running time.

Table 2.2: Summary of Improvement Scenarios

Scenario	Vehicle	Max. Speed (MPH)	Station Stops	Run Time (min)	Time Reduction (min)
D1	Diesel trainset	79	4	84.8	0.0
D2	Diesel trainset	110	4	76.3	8.5
D1-E	Diesel trainset	79	0	74.8	10.0
D3	Diesel trainset	110	4	74.2	10.6
D2-E	Diesel trainset	110	0	64.2	20.6
E1	EMU	220	4	64.0	20.8
E2	EMU	220	4	60.7	24.1
D3-E	Diesel trainset	110	0	60.3	24.5
E1-E	EMU	220	0	51.5	33.4
E2-E	EMU	220	0	47.1	37.7

The run time computed in scenario D1 represents the baseline configuration and corresponds closely with the minimum run time of 84 minutes determined from the public Amtrak timetable. The slightly longer time computed by the TPC may be attributed to differences between the actual and assumed train performance characteristics, as well as station dwell times. These results suggest that the spreadsheet-based TPC is reasonably accurate in computing running times for an intercity passenger rail service.

Evaluating the various improvement scenarios shows that there is not a substantial difference in running time between some of the express scenarios that feature a diesel locomotive consist compared to scenarios that consider an EMU and the construction of a dedicated high–speed line. The short length of the route, combined with frequent station stops limits the distance that a high-speed trainset can cruise at maximum speed. Scenario E2-E has the greatest improvement potential with a 37.7 minute reduction in the minimum run time. This scenario considers an express EMU operating over both the dedicated high-speed segment and an adjacent shared track 110-MPH segment. As illustrated in Appendix Figures C.7 through C.10, the EMU trainset does not cruise at maximum speed on a large portion of the dedicated high-speed segment. Different EMU trainsets with faster acceleration performance could be evaluated to determine their effect on further running time reductions. Alternatively, the dedicated line segment could be designed for a lower maximum speed to reduce construction costs while only marginally impacting running time.

2.5 Future Work

To date, the TPC spreadsheet has been developed primarily to assess the performance of passenger trains. With some additional processing of input parameters, the same spreadsheet could also be used to analyze the performance of freight trains. Freight trains over a mile long and weighing over 10,000 tons are typical of the North American freight rail industry. In the existing spreadsheet, these trains would span several segments with different operating speeds. Several simplifying assumptions on the TPC spreadsheet model that probably do not have a major effect on estimates of relatively light, short, passenger train performance are more likely to affect the accuracy of long heavy freight train performance. The TPC spreadsheet considers changes in speed from the reference point of the front of the train. For longer trains, this could lead to inaccurate running-time calculations due to trains accelerating to a higher speed before the last cars clear a slower speed segment. In order to account for this, a train-specific route segmentation may be created for use in the existing model. Figure 2.7 illustrates how a civil speed restriction would be incorporated into route segmentation for a train that is much longer than the length of segments generated by the changes in physical characteristics.

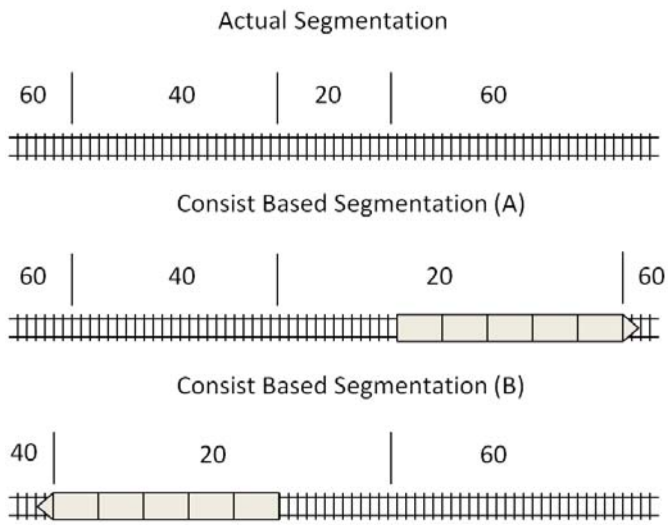


Figure 2.7: Illustration of Train-Specific Route Segmentation for use in Spreadsheet-Based TPC

Using the actual segmentation, the TPC would begin accelerating the train before the rear portion exits the 20-MPH speed restriction. In the modified segmentation schemes A and B, the train will continue at the 20-MPH lower speed until the rear end clears the limits established by the civil speed restriction. This strategy results in a different route segmentation pattern for each direction of operation and length of train.

Many commercial TPC programs include a calculation of work performed by a train consist on different segments of a route. Rail service planners and managers can use this information to determine the impact of different train consists and operating strategies on energy consumption. The spreadsheet-based TPC could be further enhanced by calculating the work performed during acceleration, cruising, and braking events within the existing script framework. The information generated by these calculations would enhance the utility of the spreadsheet for investigating various questions related to interactions between railway infrastructure and train operating costs.

2.6 Conclusions

This chapter demonstrates the functionality of a spreadsheet-based train performance calculator for use in analyzing railway improvement scenarios. The TPC can quickly and accurately compute the running time of a train given various infrastructure and rolling stock physical characteristics with the convenience and economy of ubiquitous spreadsheet software.

CHAPTER 3: INCREASING PASSENGER TRAIN SPEEDS THROUGH CURVE REALIGNMENT AND ROLLING STOCK IMPROVEMENTS

3.1 Introduction

In recent years there has been an increased focus on improving the performance of intercity passenger rail service in the United States. Introducing passenger services with greater than 79 MPH maximum speed is one approach to reducing running times; however, these types of higher-speed improvements are not always ideal on lines where track is shared with heavy-axle-load freight traffic. In addition, existing rail alignments with numerous high-degree curves may further constrain higher maximum train speeds. In order to reduce running times on lines with these constraints, curves may be re-aligned to support higher speeds. Alternatively, rolling stock with better curving performance may also be introduced to further increase train speeds.

The amount of potential curve reduction and associated speed benefit is dependent on the existing curve geometry and right-of-way characteristics. Optimal curve re-alignment design involves both the central, circular curve and the spirals that transition to the tangents at each end. A design that maximizes the speed improvement to passenger traffic must take into account the impacts of degree of curvature, angular deflection, superelevation, and cant deficiency on the spiral geometry. Changes to spiral geometry will also shift the alignment of the circular curve, even for identical degrees of curvature. The curve geometry design process is more complicated for lines operating different types of traffic whose operating characteristics are best accommodated by different curve designs. Rail wear on curves is directly related to the cumulative tonnage of traffic as well as the condition of unbalance or overbalance resulting from a difference in traffic speeds. Therefore, for reasons of economy, curve superelevation is typically designed for the predominant traffic type operating on the line (Hay 1982).

Consequently, there are sizable differences in the potential curve speed improvements when considering either a predominantly freight or passenger traffic corridor.

This chapter describes relationships between existing conditions and the maximum potential curving speed benefit for different improvement alternatives on predominantly freight or passenger railway lines. Using this research, planners and engineers of passenger rail systems can gain a better understanding of the speed improvement benefits that can be expected when upgrading an existing railway line, which in turn can be used to determine the relative cost effectiveness of different projects.

3.2 Literature Review

Several individuals and research groups have investigated topics related to curve re-alignment, higher railway curving speeds, and rolling stock curving speed improvement and testing. Their work is reviewed in the following section.

Esea (1991) presented a railway curve realignment model that uses the string-lining method to smooth out irregularities in track geometry. The model is formulated as a mixed integer program that minimizes the total positive and negative deviations between a realigned and ideal curve at all stations. The paper presents a practical and efficient methodology of correcting curve alignment deviations, but is not suitable for re-aligning a curve to support higher speeds.

Lombardi (1994) summarized the results of the operation of high-speed test trains on Amtrak's Northeast Corridor (NEC) for the years 1992 and 1993. In these test runs two types of international rolling stock were operated on the existing NEC track structure: a German ICE non-tilting train and a Swedish X2000 tilting train. Part of the test program included assessing the curving performance of both trains. Test runs were conducted with instrumented wheel sets and rolling stock accelerometers at up to 125 MPH at cant deficiencies up to 12 inches for the X2000 and 7 inches for the ICE. The tests demonstrated the curving performance of the X2000 tilting train and were a successful demonstration of international, high-performance intercity passenger rolling stock in the United States.

Andersson et al. (1995) outlined the development and testing of the X2000 trainset for service on the Swedish State Railways. This type of tilting rolling stock was developed to increase average speed by 30-35% over conventional trains without making substantial changes to the alignment of the conventional rail infrastructure. A premium truck design and active carbody tilting system enables the train to travel at speeds of up to 125 MPH at cant deficiencies corresponding to 1.6 m/s^2 (approximately 0.16g) uncompensated lateral acceleration as measured parallel to the track plane. This figure corresponds with a cant deficiency of approximately 9”. The authors conclude that the X2000 enabled higher operating speeds on existing curved tracks while minimizing the investment in the infrastructure. The paper makes no mention of modifications to spiral easement curves prior to the introduction of high cant-deficiency operations.

Tsai et al. (1995) outlined the testing procedure necessary to implement higher curving speeds for rolling stock on existing rail lines. The authors state that the most economical approach to increasing average speeds on existing lines is to improve braking performance and maximum allowable cant deficiency in curve segments. The use of lateral and vertical accelerometers mounted on a vehicle carbody floor was the main technique outlined for evaluating a known vehicle type for ride quality and cant deficiency performance on new routes.

Andersson et al. (1996) described the radial self-steering trucks developed for the X2000 train to reduce track forces during high-speed curving. The paper also describes computer modeling techniques for simulating forces at the wheel rail interface. Although the carbody tilting system preserves passenger comfort on this type of rolling stock, it is the performance of the trucks controlling forces at the wheel-rail interface that allows the operation of the equipment at higher cant deficiencies. The authors found that the model results were generally within 10% of values measured in field testing.

Kolig and Hesser (1997) described the design features for the type of tilting technology incorporated into the *Acela Express* trainset that was under development for use on Amtrak's NEC. The design requirement for the trainset was the capability of up to 12 inches of cant deficiency, with 9 inches to be used in revenue service. In addition, lateral acceleration experienced by the passenger had to be limited to less than 0.1g or approximately $0.98 m/s^2$. A tilting system was necessary to avoid exceeding acceleration limits while travelling at high speeds.

Harris et al. (1998) provided an introduction to the general physics of train curving behavior with tilting rolling stock. As mentioned above, this type of rolling stock is used to compensate for lateral forces experienced by passengers at speeds where there is insufficient cant in curves to achieve this otherwise. The authors highlight the importance of the curve spirals in allowing the rolling stock tilting mechanism to gradually increase the degree of carbody inclination necessary to satisfy passenger comfort. The authors point out that tilting technology effectively changes the limiting factor in curving speed from passenger comfort criteria to safe limits of vehicle stability and the state of forces at the wheel rail interface.

O'Dwyer (1997) described a versine-based procedure for calculating lateral displacements to correctly realign a railway curve. O'Dwyer shows that a proposed realignment must satisfy two constraints. The sum of the design versines must equal that of the existing versines and also the centroid of the proposed realignment must match the centroid of the original alignment. The paper describes the use of linear programming for this problem type and describes several potential formulations. The author does not mention the model proposed by Esea (1991) although both models address the problem of curve realignment.

Lombardi et al. (2002) provided an overview of the types of tests that were necessary to obtain a waiver from the Federal Railroad Administration (FRA) to operate rolling stock at high cant deficiencies at speeds less than 90 MPH. The testing process for MARC-III bilevel cars, the *Acela Express* rolling stock, and the Amtrak *Cascades* Talgo rolling stock (Figure 3.1) is outlined in this work. At that time, there were no FRA regulations for operating rolling stock at greater than four inches of cant deficiency on track classes one through five. The description of the testing of the Talgo equipment is unique in that it is the only example of tilting equipment in regular operation on a U.S. Class I freight railroad.

Figure 3.1: Talgo Tilting Trainset Operating in Amtrak *Cascades* Service (Wilson 2010)

Carbody lateral acceleration instrumentation was applied to the passenger cars and the locomotives at each end of the trainset. Additional instrumentation was placed in curves to monitor truck side and single wheel lateral and vertical forces applied to the rail (L/V ratios). The tests showed that the Talgo equipment was safe to operate at cant deficiencies up to 8 inches; however, the locomotives exceeded lateral acceleration limits at these levels. Based on these tests, the FRA granted a waiver to operate the Talgo equipment at six inches of cant deficiency, with a revenue service cant deficiency established by the BNSF Railway at 5 inches, as an extra factor of safety.

Although not stated by Lombardi et al, (2002) there are examples of conventional, non-tilting equipment that operates in revenue service at five inches of cant deficiency. On the *Cascades* service, the potentially greater speed benefit of the tilting equipment cannot be realized due in part to the limitations of the curving behavior of the locomotives used on the route. As part of the waiver granted by the FRA, carbody accelerometers are used to measure steady state and peak-to-peak lateral accelerations on a quarterly basis.

Kufver (2005) analyzes track components and geometry with the operation of high cant deficiency passenger trains. The author states that modern track components are generally capable of carrying the increased lateral and vertical loads caused by high cant deficiency operation. The author notes that the alteration of track alignment to support higher speeds has the competing objectives of increasing the length of transition curves and increasing the radius of the curve. The optimal solution for some curves might therefore include an increased length of transition curve and a reduction in curve radius in order for higher speed modifications to existing on an existing right-of-way footprint. The author recommends that the European

Committee for Standardization (CEN) adopt a limit rate of change of cant deficiency based on lateral acceleration, lateral jerk, and roll velocity.

Marquis (2011) outlined simulation work using the NUCARS software performed for the FRA in order to study the performance of rail vehicles at speeds greater than 90 MPH and cant deficiencies greater than five inches. In this study, an *Acela* power car and an Amfleet passenger coach were modeled, and the results compared to field response data collected for both rolling stock types. The results of this comparison were used to support proposed modifications of FRA track geometry standards. An interesting result of the analysis was that some of the allowable track geometry deviations were higher for higher speeds and lower cant deficiencies than for high cant deficiencies at comparatively lower speeds. The results suggest that in some cases high cant deficiency operation results in conditions of greater concern than higher speed operation. The authors conclude that further research is needed to understand how to design trucks to improve the performance of vehicles operating at high cant deficiency.

Lai and Po-Wen (2012) presented a framework for using mathematical programming to identify an optimal strategy to reduce running time on a passenger rail corridor. The problem is formulated as a mixed integer program that maximizes the reduction in travel time for different combinations of rolling stock and infrastructure. This model considered the performance of various trains, including several types of rolling stock with varied curving performance. The model also considered a specific set of line improvements, but given the long length of segments did not account for interaction effects between adjacent route segments.

FRA (2013a) issued a final rule on proposed changes to track safety standards and passenger equipment safety standards in 2013. As part of this rule change, procedures were added to qualify rolling stock for high cant deficiency operation on FRA track classes 1 through

5 without obtaining a waiver. Prior to this change, passenger operations such as the Amtrak *Cascades* service had to undergo a lengthy testing and approval process to introduce higher cant deficiencies. Because marginal running time benefits per unit increase in velocity are greater when upgrading a given length of lower-speed, compared to the same length of higher-speed track, this waiver procedure hindered the implementation of higher cant deficiency operation at the speeds where it was potentially the most beneficial. The 2013 rule outlined the testing requirements necessary to approve higher cant deficiencies on track classes 1 through 5. In addition, rolling stock that is already qualified to operate at 3 to 5 inches of cant deficiency will be considered qualified to operate at its permitted cant deficiency for any track segment. This change in regulation has the potential to reduce running times of existing and proposed passenger service, while preserving safe operation.

Dick et al. (2016) developed a framework for selecting the optimal combination of actual superelevation and cant deficiency for shared-corridor curves with a distribution of passenger and freight train speeds. Graphical and mixed-integer programming approaches are used to maximize the speed of passenger trains while minimizing the number of freight trains traveling below the balancing speed.

3.3 Background of Curve Geometry and Train Speed

The maximum operating speed on railway curves is given by Equation 3.1 (FRA 2013b). In this equation, V_{max} is the maximum operating speed in MPH. E_a and E_u are the actual curve superelevation and the unbalanced elevation or cant deficiency to equilibrium condition respectively. The parameter D is the degree of curve. The following section describes the geometric parameters of railway curves that most influence maximum operating speed.

$$V_{max} = \sqrt{\frac{E_a + E_u}{0.0007D}} \tag{3.1}$$

Degree of Curve (D) – In U.S. railway practice, the degree of curve is defined as the angle subtended by a chord of 100 ft (Hay 1982; ARMEA 2010b). The degree of curve indicates the sharpness of curve and is inversely related to curve radius by Equation 3.2, and can be roughly approximated by Equation 3.3 for low degree curves. Lower degrees of curve correspond to a larger radius and allow for higher train operating speeds. Curves of 1° - 5° are common for high density freight railroads and do not generally limit the speed of freight traffic. These curves, however, may present a barrier to the operation of higher-speed passenger traffic.

$$R = \frac{50}{\sin\left(\frac{D}{2}\right)} \tag{3.2}$$

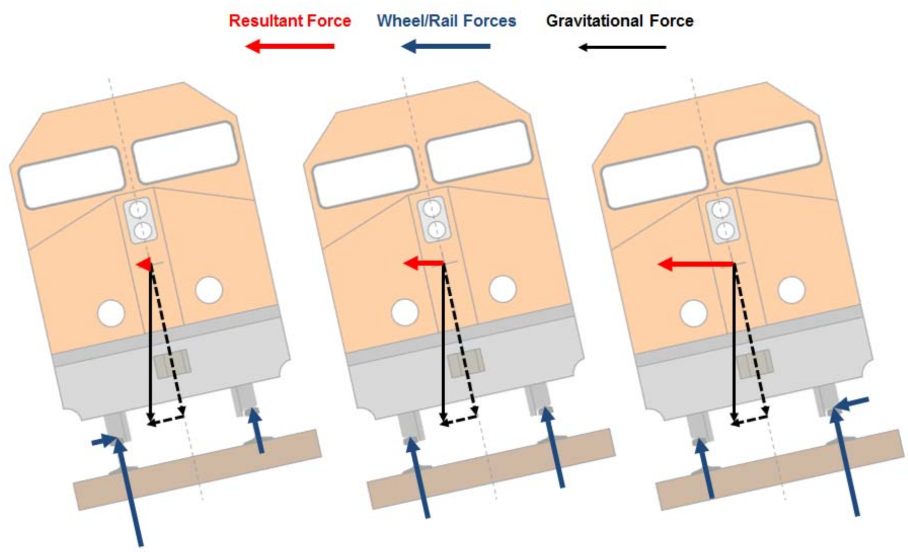
$$R \sim \frac{5730}{D} \tag{3.3}$$

Superelevation (E_a) – Superelevation is the difference in elevation between the high and low rails on a curved segment of track. In North American practice superelevation is expressed in inches. For a given train operating speed and degree of curve, there is an amount of superelevation for which track plane lateral forces are compensated for by the lateral component of gravitational force introduced by the inclination of the track structure. This amount of superelevation is called the *equilibrium elevation* and the corresponding vehicle speed is called the *equilibrium speed*. For reasons discussed previously the majority of the North American routes use superelevation designed for freight rather than passenger traffic.

Cant deficiency (E_u) – A cant deficiency exists when a vehicle travels through a curve at a speed greater than its equilibrium speed. The difference between the actual superelevation and the superelevation required to compensate for the track plane lateral forces at the higher train speed is the cant deficiency, usually expressed in inches. Railroads may intentionally design curve superelevation with an amount of cant deficiency for the predominant traffic type. As an example, the BNSF Railway (2010) designs curve superelevation with a 2 inch cant deficiency for freight traffic. Up to a point, traffic operating at a higher speed than the predominant traffic is allowed to operate with a cant deficiency. FRA (2013b) regulation permits operation of all types of rolling stock at up to 3 inches of cant deficiency. Rolling stock that meets certain additional regulatory requirements is permitted to operate at higher cant deficiencies (FRA 2013a).

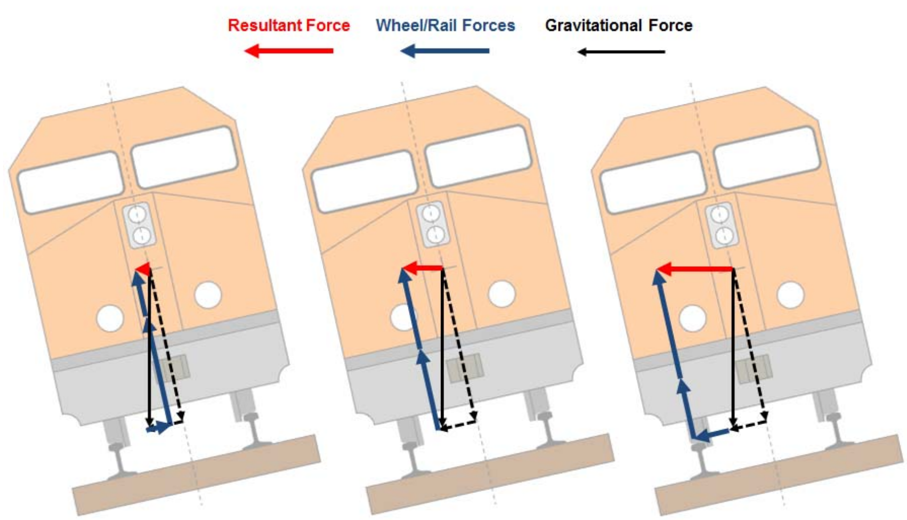
Cant excess – A cant excess or overbalance condition exists when a vehicle travels through a curve at a speed less than the equilibrium speed of that curve. The difference between the actual superelevation and the superelevation required for equilibrium conditions at the lower speed is the cant excess, again usually expressed in inches. Operation of the predominant traffic in cant excess conditions can lead to higher magnitude low rail forces (Kerchoff 2012). These forces can cause excessive low-rail wear and an elevated risk of rail rollover derailments. Curve superelevation is therefore typically designed with some amount of cant deficiency for the predominant traffic type so as to avoid greater amounts of cant excess when traffic must, for various reasons, operate at less than maximum speed.

Discussion of curve operating conditions - Cant excess, equilibrium, and cant deficiency operating conditions are illustrated in Figure 3.2. In this diagram, the different forces acting on a rail vehicle are shown. The red arrow acting on the vehicle mass center is the net resultant force that is accelerating the vehicle around the curve. This resultant is the sum of all the other forces illustrated. The black arrow in each scenario is the force due to gravity that acts on the vehicle mass center. The dashed black lines illustrate vertical and lateral components of the gravitational force acting on the track structure. The blue arrows represent the vertical and lateral forces acting on the rail vehicle at the wheel rail interface. At equilibrium speed, the high and low-rail vertical forces are equal in magnitude and there is no lateral force at the wheel-rail interface. In an overbalance condition (Figure 3.2A) the low-rail vertical force is greater in magnitude than high-rail vertical force. In this condition, there is also lateral force acting on the rolling stock from the low-rail. In an underbalance condition (Figure 3.2C) the high-rail vertical force is greater in magnitude than that of the low rail. In addition, there is a lateral force that acts on the rolling stock from the high rail. Figure 3.3 shows the corresponding sets of vector additions of forces under different operating conditions. The resultant force guides the rail vehicle in a circular path around the curve.



A) Overbalance ($V < V_e$) B) Equilibrium ($V=V_e$) C) Underbalance ($V>V_e$)

Figure 3.2: Illustration of Forces Acting on Rolling Stock in Different Curve Operating Conditions



A) Overbalance ($V < V_e$) B) Equilibrium ($V=V_e$) C) Underbalance ($V>V_e$)

Figure 3.3: Illustration of Rolling Stock Force Vector Addition in Different Curve Operating Conditions

On passenger trains operating around a curve in a cant-deficiency condition, passengers experience a centrifugal force towards the outside of the carbody due to the acceleration of the their perceived reference frame towards the inside of the curve. At high cant deficiencies this force can affect the comfort and sometimes safety of passengers, even if the rolling stock is well below thresholds for derailment or overturning. In these cases, premium rolling stock designs such as tilting trains can preserve passenger comfort by compensating for these high lateral forces. In a tilting train, the carbody rotates toward the inside of the curve, increasing the magnitude of the lateral component of the passenger gravitational force that acts in the opposite direction to that of the perceived centrifugal force. Although tilting equipment preserves passenger comfort, it has no impact on the forces at the wheel rail interface and does not compensate for the increased high rail lateral force introduced by high cant deficiency operation (Lombardi, 1994, Klauser 2005).

Length of Spiral **–** Spiral or easement curves are used on mainline track as a transition from tangent track to circular curves in order to gradually change superelevation in correspondence with the curve radius. The spiral easement must be sufficiently long so as not to exceed the allowable time or distance rate of change of superelevation, nor should the length produce conditions that exceed the allowable time rate of change in cant deficiency. Longer spirals with lower rates of change of superelevation and cant deficiency enhance passenger comfort by limiting the rate of change of lateral acceleration and carbody roll. Recommended practice for design spiral length varies considerably on different railways around the world. In the following section I compare typical North American spiral design standards to those in use in the United Kingdom (U.K.) and Sweden, two examples representative of international practice on shared-use freight and passenger trackage.

The American Railway Engineering and Maintenance-of-Way Association (AREMA) outlines three formulae for determining the length of a spiral easement. Equations 3.4 and 3.5 are typically used together to determine the minimum length of spiral for a given curve. In these equations, L is expressed in feet, E_u and E_a in inches, and V in miles per hour. AREMA recommends using equation 3.6 instead of equation 3.4 for locations where the former would result in an uneconomical spiral length. Equations 3.4 and 3.6 implicitly limit the rate of change in cant deficiency to 0.90 in/sec and 1.20 in/sec respectively.

$$L = 1.63E_uV \tag{3.4}$$

$$L = 62E_a \tag{3.5}$$

$$L = 1.22E_uV \tag{3.6}$$

Table 3.1 compares the curve and spiral design parameters of North American practice to those of the U.K. and Sweden. Examples of North American practice include AREMA recommended practice as well as design practice from Union Pacific (UP 2013) and Canadian National (CN 2002) railroads. There is considerable variation in both domestic and international spiral design characteristics. Most international standards, in addition to the UP, prescribe a maximum limit to the time rate of change of superelevation. In addition, most sets of standards outline a maximum time rate of change of cant deficiency. While AREMA practice limits the time rate of change of cant deficiency, the allowable rate of change is higher in both British and Swedish practice. As an example, conventional traffic in Sweden is limited to a rate of change of cant deficiency of 1.81 in/sec, considerably higher than the 1.20 in/sec that is implicit in equation 6.

Table 3.1: Comparison of Design Standards

Parameter	FRA / AREMA	UP	CN RM 1305-0	UK Normal[1]	UK Exceptional[1]	Sweden (A)[1]	Sweden (S)[1]
Max. E_a (in)	7.00	5.00	5.00	5.91	7.09	5.91	5.91
Max. E_u (in)	3.00+	n/a	6.00	4.33	5.91	3.94	9.65
Time Rate of Change of E_a (in/sec)	n/a	*	n/a	1.38	3.34	1.81	2.76
Time Rate of Change of E_u (in/sec)	0.90	*	0.90	1.38	2.76	1.81	3.11

[1] Kufver (2005)
* See table 3.2

Table 3.2 illustrates the spiral length design practice for the UP. For different speeds, UP outlines a design value for the amount of runoff distance in feet for a one-inch change in superelevation for different speeds. Using representative values for superelevation and UP practice of designing curve superelevation for one-inch cant deficiency for freight traffic, the time rate of change for E_a and E_u can be determined at different speeds. The table illustrates that UP practice has a variable time rate of change of E_a and E_u that depends on operating speed. The time rate of change of superelevation dE_a/dt converges to 1.25 in/sec for the highest speeds outlined. The time rate of change of cant deficiency dE_u/dt also converges to a constant value at higher speeds; however, this value is dependent on the amount of curve superelevation.

Table 3.2: Union Pacific Railroad Indicated Time Rates of Change of E_a and E_u

		E_a = 2"				E_a = 4"			
V (MPH)	dx/dE_a (ft/in)	100(S) (ft)	dt (sec)	dE_a/dt (in/sec)	dE_u/dt (in/sec)	100(S) (ft)	dt (sec)	dE_a/dt (in/sec)	dE_u/dt (in/sec)
20	44	88	3.00	0.67	0.33	176	6.00	0.67	0.17
25	44	88	2.40	0.83	0.42	176	4.80	0.83	0.21
30	44	88	2.00	1.00	0.50	176	4.00	1.00	0.25
35	44	88	1.71	1.17	0.58	176	3.43	1.17	0.29
40	47	94	1.60	1.25	0.62	188	3.20	1.25	0.31
45	53	106	1.61	1.25	0.62	212	3.21	1.25	0.31
50	59	118	1.61	1.24	0.62	236	3.22	1.24	0.31
55	65	130	1.61	1.24	0.62	260	3.22	1.24	0.31
60	70	140	1.59	1.26	0.63	280	3.18	1.26	0.31
65	76	152	1.59	1.25	0.63	304	3.19	1.25	0.31
70	82	164	1.60	1.25	0.63	328	3.19	1.25	0.31
75	88	176	1.60	1.25	0.63	352	3.20	1.25	0.31
80	94	188	1.60	1.25	0.62	376	3.20	1.25	0.31
85	100	200	1.60	1.25	0.62	400	3.21	1.25	0.31
90	106	212	1.61	1.25	0.62	424	3.21	1.25	0.31

[1] UP (2013)

Tables 3.3 and 3.4 show spiral lengths calculated using the seven different groups of standards and recommended practices. Table 3.3 shows spiral lengths calculated for a design speed of 79 MPH while Table 3.4 shows those calculated for 110 MPH. For these tables, input values of 2, 4, and 6 inches of superelevation were used in combinations with 3, 5, and 9 inches of cant deficiency. These values were selected because they are representative of conditions on various North American railway lines with either passenger or freight as the predominant traffic type. For example, many Class I freight railroads limit superelevation to a maximum of four inches (BNSF 2010). Three inches of cant deficiency is a common maximum value for contemporary intercity rail lines although cant deficiencies of five inches may become more common following the recent revision of the FRA track safety standards. The tables shown do not generally extend the spiral design criteria beyond the maximum limits prescribed by their accompanying standards. Blank cell values indicate an invalid combination of input parameters for the design practice used.

Table 3.3: Spiral Length Comparison for V = 79 MPH (values in feet)

	E_a = 2"			E_a = 4"			E_a = 6"		
E_u (in)	3	5	9	3	5	9	3	5	9
AREMA (1.2 in/sec)	289	482	867	289	482	867	372	482	867
UP	185			371					
CN RM 1305-0	386	386		289	289				
UK Normal	252	420		336	420		504	504	
UK Exceptional	126	210		139	210		208	210	
Sweden (A)	192	320		256	320		384	384	
Sweden (S)	112	186	335	168	186	335	252	252	335

Table 3.4: Spiral Length Comparison for V = 110 MPH (values in feet)

	E_a = 2"			E_a = 4"			E_a = 6"		
E_u (in)	3	5	9	3	5	9	3	5	9
AREMA (1.2 in/sec)	403	671	1,208	403	671	1,208	403	671	1,208
UP	258			516					
CN RM 1305-0	538	538		403	403				
UK Normal	351	585		468	585		701	701	
UK Exceptional	175	292		193	292		290	292	
Sweden (A)	267	446		357	446		535	535	
Sweden (S)	156	259	467	234	259	467	351	351	467

Higher operating speeds demand correspondingly longer spiral lengths for all sets of standards. At high cant deficiencies the AREMA methodology can result in spirals longer by roughly a factor of three over the standards for Swedish tilting trains. This illustrates the wide range in spiral design practice and suggests that the AREMA methodology may be conservative for some scenarios. When adapting an existing line for high cant deficiencies, the AREMA method is likely to considerably increase the costs of shifting curves and/or limit the maximum potential speed benefit of curve realignments with premium, high-cant-deficiency rolling stock designs.

3.4 Method for Determining Maximum Speed Improvement

Many North American railway lines were not designed to support the kind of sustained high speeds necessary for intercity passenger service to achieve running times that are competitive with other modern transportation modes. On lines where curves frequently limit the speed of the rail service to less than a design maximum speed, there may be opportunities to improve curving speeds and reduce overall running time through curve realignment projects. The speed benefit of these projects can be increased further when combined with rolling stock designs capable of safe and comfortable operation at high cant deficiencies. The following section discusses the relationship between various infrastructure and train operating parameters with the maximum speed benefit that could be expected with various improvement strategies.

Three approaches to increase curving speed are: increase superelevation, increase maximum allowable cant deficiency, or reduce the degree of curvature. The strategy for changing these parameters is different depending on whether the line has predominantly freight or passenger traffic. On lines dominated by passenger traffic, superelevation and spiral lengths can be increased to the maximum allowable limits in order to allow for higher speeds. It should be noted that increasing the length of spirals requires a shift of curve alignment toward the inside of the right-of-way for the same degree of curve. If the curve in question is already at the maximum allowable limit for superelevation, then the curve can be shifted further to the inside of the right-of-way in order to reduce the degree of curvature. The amount of potential shift depends greatly on the width of right-of-way and other track structure characteristics at different points along the curve. For example, if the curve is situated on a high fill, the maximum allowable curve shift is less than a similar curve on a low embankment for the same width right of way.

Figures 3.4 and 3.5 illustrate the realignment of a curve to reduce the degree of curve and/or increase the length of spirals. Figure 3.4 shows the original curve alignment, with point of spiral (PS), spiral curve (SC), curve spiral (CS), and spiral-tangent (ST) points noted. The curve has a central angle *I* and a right-of-way width *W*. As the degree of curve and spiral length are modified to allow for increased train speeds, the alignment is shifted toward the inside of the curve (Figure 3.5). Figure 3.6 shows further enhancement of the curve geometry such that the alignment exceeds the footprint of the existing right of way. This type of project may be cost prohibitive, depending on the potential presence of natural or manmade obstacles adjacent to the railway line. The methodology for evaluating a curve shift that transcends the ROW boundary is the same as that which considers a ROW constrained shift. When planning curve improvements, ROW acquisition can greatly increase costs, schedule duration, and complexity of environmental permitting and public relations efforts. It is therefore important to understand the maximum speed improvement that can be realized on a curve without transcending the ROW boundary.

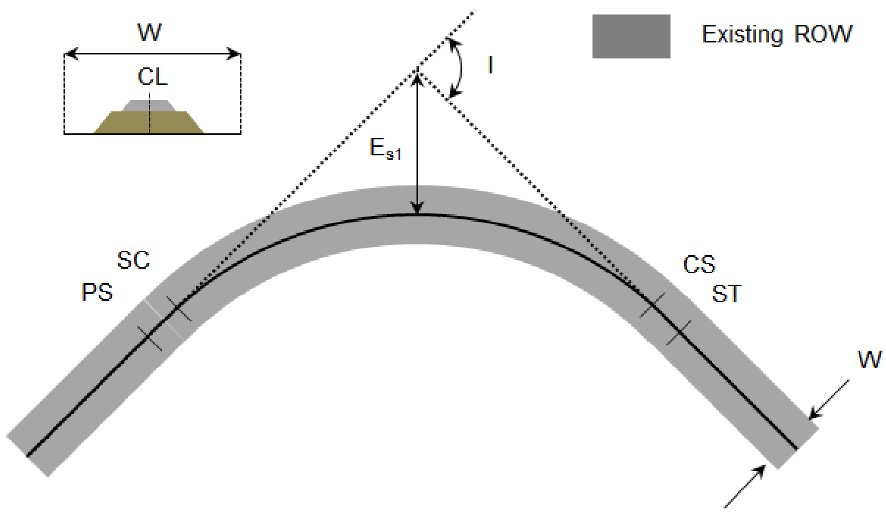


Figure 3.4: Example Existing Curve Geometry

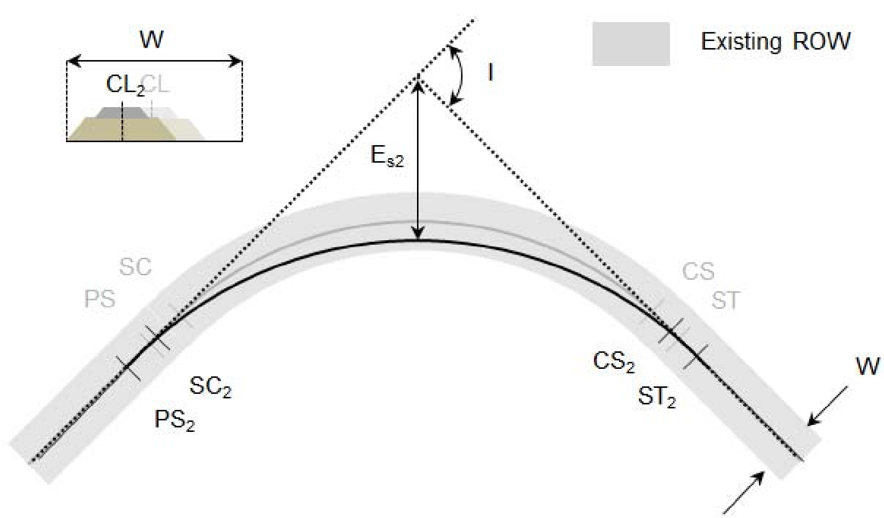


Figure 3.5: Enhanced Curve Geometry on Existing ROW

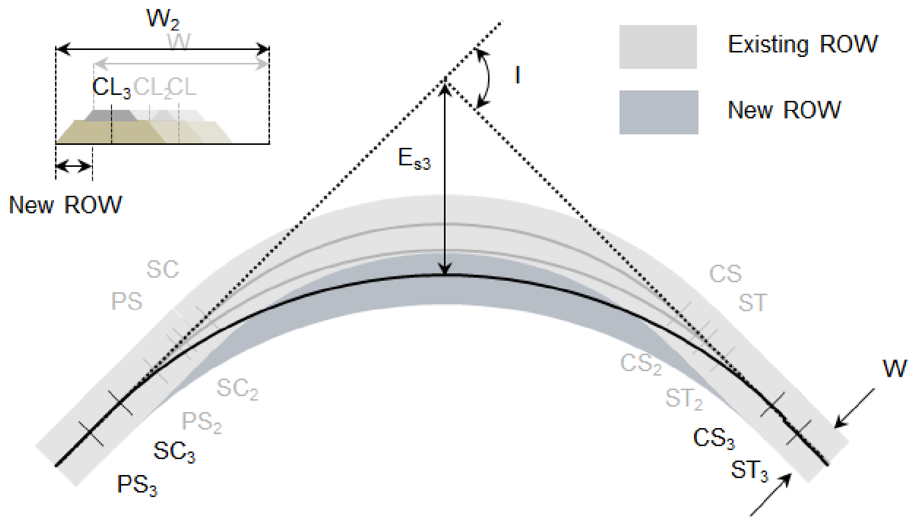


Figure 3.6: Enhanced Curve Geometry on Expanded ROW

Improving passenger train curving speeds on a line with predominantly freight traffic carries the added complication that the superelevation is designed for freight train speed and cant deficiency. In this scenario, passenger traffic speeds can be increased by decreasing the degree of curve and shifting the alignment to the inside of the right-of-way. The speed benefit of this shift is less than the corresponding shift in a line dominated by passenger traffic because the superelevation on the freight dominated line must be reduced along with degree of curve. For the analysis presented in this work, the procedure for re-aligning a curve on a freight railway line is as follows.

1. Determine existing curve geometry including degree of curve, deflection angle, length of spiral, and speeds of freight and passenger traffic.
2. Determine future design speed of freight and passenger traffic, design cant deficiency for freight traffic, and maximum allowable cant deficiency for passenger traffic

3. The passenger train speed, assuming adequate spiral conditions, is given by Equations 3.7, 3.8 and 3.9. The equation used depends on the amount of superelevation on the curve, and is governed by freight traffic speed, degree of curve, and design unbalance. Equation 3.7 is used for curves where the freight traffic does not require any superelevation. Equation 3.8 is used for curves that have an amount of superelevation greater than zero but less than the maximum allowable. Equation 3.9 is used for curves that have the maximum allowable superelevation. In some low degree curves, the passenger train speed V_p can also be limited by the maximum speed of the rolling stock. For cases where equations 3.7, 3.8 or 3.9 result in a value greater than the maximum rolling stock speed, the proposed curve shift benefit will not be fully realized.

$$V_p = \sqrt{\frac{E_{u,p}}{0.0007D}} \tag{3.7}$$

$$V_p = \sqrt{\frac{(0.0007V_f^2 D - E_{u,f}) + E_{u,p}}{0.0007D}} \tag{3.8}$$

$$V_p = \sqrt{\frac{E_{a,max} + E_{u,p}}{0.0007D}} \tag{3.9}$$

4. The final degree of curve that is feasible for a given track center shift depends on the length of spirals and is usually governed by the speed of passenger traffic. As the

degree of curve decreases, the allowable passenger train speed increases and in turn requires a longer transition curve. Spiral easement length can be determined by equations 3.4, 3.5 and 3.6 when using AREMA practice. Alternatively, the spiral length can be determined using the maximum allowable rates of change of superelevation and cant deficiency. Equations 3.10, 3.11, and 3.12 show the relationship between spiral length (S) in hundreds of feet with E_a and E_u in inches, dE_u/dt in inches per second, dE_a/dt in inches per second, dE_a/dx in inches per foot, and finally passenger train speed V_p in miles per hour. The maximum spiral length produced by these equations is carried forward in the calculations.

$$S = \frac{E_u}{\frac{dE_u}{dt}} \times 0.01467V_p \tag{3.10}$$

$$S = \frac{E_a}{\frac{dE_a}{dt}} \times 0.01467V_p \tag{3.11}$$

$$S = 0.01\frac{E_a}{\frac{dE_a}{dx}} \tag{3.12}$$

5. Determine the maximum allowable shift, *A*, in the center line of curve at the middle of the curve body
6. Calculate external distance E_s from existing curve center line to the point of intersection *PI* of the tangents

$$E_s = (R + o)\text{exsec}\left(\frac{I}{2}\right) + o \tag{3.13}$$

7. Add external distance to allowable curve shift to determine new curve external distance. Substituting Equations 3.14, 3.15 and 3.16, the new external distance is then related to spiral length and degree of curve by Equation 3.17. In these equations, *R* is the curve radius in feet, *I* is the deflection angle in degrees, *D* is the re-aligned degree of curvature, and *S* is the re-aligned transition curve length hundreds of feet.

$$o = 0.1454\Delta S \tag{3.14}$$

$$\Delta = \frac{1}{2}kS^2 \tag{3.15}$$

$$k = \frac{D}{S} \tag{3.16}$$

$$E_s + A = \frac{5730}{D}\text{exsec}\left(\frac{I}{2}\right) + 0.0727DS^2\text{exsec}\left(\frac{I}{2}\right) + 0.0727DS^2 \tag{3.17}$$

8. Solve equation 3.17 for the re-aligned degree of curve substituting for S with the formulas derived in equations 3.10 – 3.12.

I found that substituting the expressions for *D* and *S* into equation 3.17 does not yield an equation with a convenient closed form symbolic solution. Fortunately, it is easy to determine a numerical solution for degree of curve using readily available spreadsheet software. Using Microsoft Excel and a Visual Basic for Applications script, an entire route of curves can be analyzed in minutes to determine the maximum potential speed benefit in different rolling stock and operating scenarios.

3.5 Maximum Single Curve Speed Improvement

The maximum potential speed improvement for a curve realignment can be determined with existing geometry as well as present and future operating parameters. Using the previously described methodology, a set of curves with different deflection angles and degrees of curve were analyzed for their potential speed improvement. The curves selected are representative of a wide range of potential curve geometries; however, the same analysis could also be performed to the specific geometry of each curve along an existing railway line. The purpose of illustrating the potential improvement on the representative set of curves is to give the reader an idea of the potential benefit that can be derived from these projects, as well as the implications of different corridor operating scenarios. The two main sets of input parameters are illustrated in Table 3.5. All scenarios were constrained by a maximum allowable 15-foot shift in track centerline at the middle of the curve body.

Table 3.5: Parameters for Speed Improvement Scenarios

Parameter	Freight Corridor	Passenger Corridor
Maximum superelevation	2,4 inches	2,4,6 inches
Freight traffic speed	70 MPH	70 MPH
Initial passenger traffic speed	79 MPH	79 MPH
Traffic governing curve superelevation	Freight	Passenger
Design cant deficiency (traffic type)	2 inches (freight)	3 - 9 inches (passenger)
Initial passenger cant deficiency	3 inches	3 inches

The methodology for determining spiral length is another important input for each scenario. As stated earlier, spiral length criteria varies widely in both domestic and international practice. For this analysis, spiral lengths were determined using Equations 3.5 and 3.6 for cant deficiencies up to 5 inches. For 5 inches of cant deficiency or greater, spiral length was designed by taking the longest spiral length produced by either the AREMA equations for a conventional passenger train at 5 inches of cant deficiency, or the length resulting from Swedish criteria for tilting trains at the highest level of cant deficiency considered in the scenario.

Figures 3.7 – 3.12 illustrate the potential curve speed improvement for a series of different initial and modified operating scenarios. The speed improvement in each scenario is calculated against a baseline passenger speed assuming an initial 79 MPH maximum equipment speed and 3 inches of maximum cant deficiency. In this analysis, both the initial and final curve geometry had to contain the greater of either: 100 ft of circular curve, or the equivalent length of circular curve covered by three seconds of running time at the maximum operating speed. Each cell value in the table reflects the higher speed and the initial speed in MPH with the cells color coded based on the magnitude of improvement. The blue and white borders define the separation between scenarios with different allowable maximum design values for superelevation. For some combinations of initial degree of curve and deflection angle, lower design values of superelevation increase the scope of feasible curve geometries due to lower spiral length requirements. The degree of curvature D_o is indicated on the vertical axis, and the curve deflection angle I on the horizontal. In each table, black cells indicate curve-geometry combinations that are infeasible due to the calculated length of circular curve and spirals.

Figures 3.7, 3.8, and 3.9 illustrate speed improvement potential for lines with predominantly freight traffic. Figure 3.7 considers curve realignment with no change in allowable passenger train cant deficiency. In this scenario, the speed improvement can be attributed to just the curve realignment. Figures 3.8 and 3.9 consider an increase from three inches of cant deficiency to five and nine inches of cant deficiency, respectively. The five-inch cant deficiency scenario represents the maximum for most conventional passenger equipment in the United States, while the nine-inch cant deficiency scenario would represent the introduction of rolling stock with tilting capabilities. In these scenarios the speed improvement benefit results from a combination of curve realignment as well as introduction of higher cant deficiency passenger traffic operation.

D_o	Deflection Angle (I) 10	15	20	25	30	35	40	45	50	55	60
0.50	107 - 79	99 - 79	96 - 79	95 - 79	94 - 79	93 - 79	93 - 79	93 - 79	93 - 79	93 - 79	92 - 79
0.75	92 - 79	85 - 79	83 - 79	83 - 79	83 - 79	82 - 79	82 - 79	82 - 79	82 - 79	82 - 79	82 - 79
1.00	85 - 79	82 - 79	81 - 79	80 - 79	80 - 79	80 - 79	79 - 79	79 - 79	79 - 79	79 - 79	79 - 79
1.25	83 - 77	80 - 77	79 - 77	78 - 77	78 - 77	78 - 77	78 - 77	78 - 77	77 - 77	77 - 77	77 - 77
1.50	82 - 76	79 - 76	78 - 76	77 - 76	77 - 76	77 - 76	76 - 76	76 - 76	76 - 76	76 - 76	76 - 76
1.75	81 - 63	78 - 75	77 - 75	76 - 75	76 - 75	76 - 75	75 - 75	75 - 75	75 - 75	75 - 75	75 - 75
2.0	81 - 59	77 - 70	76 - 70	75 - 70	75 - 70	74 - 70	73 - 70	72 - 70	72 - 70	72 - 70	71 - 70
2.5		76 - 63	75 - 63	71 - 63	68 - 63	67 - 63	66 - 63	65 - 63	65 - 63	64 - 63	64 - 63
3.0		67 - 48	71 - 57	66 - 57	63 - 57	62 - 57	61 - 57	60 - 57	59 - 57	59 - 57	59 - 57
3.5		64 - 45	67 - 53	62 - 53	60 - 53	58 - 53	57 - 53	56 - 53	55 - 53	55 - 53	55 - 53
4.0		62 - 42	65 - 50	60 - 50	57 - 50	55 - 50	53 - 50	53 - 50	52 - 50	52 - 50	51 - 50
5.0			51 - 37	56 - 44	52 - 44	50 - 44	49 - 44	48 - 44	47 - 44	46 - 44	46 - 44
6.0			49 - 34	44 - 34	49 - 40	47 - 40	45 - 40	44 - 40	43 - 40	43 - 40	42 - 40
7.0			47 - 31	42 - 31	47 - 37	44 - 37	43 - 37	41 - 37	41 - 37	40 - 37	40 - 37
8.0				40 - 29	37 - 29	42 - 35	41 - 35	39 - 35	38 - 35	38 - 35	37 - 35
9.0				39 - 28	36 - 28	34 - 28	39 - 33	38 - 33	37 - 33	36 - 33	35 - 33
10.0				38 - 26	35 - 26	33 - 26	38 - 31	36 - 31	35 - 31	34 - 31	34 - 31
11.0					34 - 25	32 - 25	30 - 25	35 - 30	34 - 30	33 - 30	32 - 30
12.0					33 - 24	31 - 24	29 - 24	28 - 24	33 - 28	32 - 28	31 - 28
13.0					33 - 23	30 - 23	29 - 23	27 - 23	32 - 27	31 - 27	30 - 27
14.0						30 - 22	28 - 22	27 - 22	26 - 22	30 - 26	30 - 26

Below Blue Line E_a = 2" - Above Blue Line E_a = 4"

0 to 5 MPH	5 to 10 MPH	10 to 15 MPH	15 to 20 MPH	20 to 25 MPH	25 to 30 MPH	30 to 35 MPH	35+ MPH

Figure 3.7: Freight Corridor Maximum Passenger Speed Improvement (MPH)

A = 15 feet, $E_{u,p,initial} = E_{u,p,final}$ = 3 inches

Deflection Angle (I)

D_o	10	15	20	25	30	35	40	45	50	55	60
0.50	136 - 79	127 - 79	123 - 79	122 - 79	121 - 79	120 - 79	120 - 79	120 - 79	120 - 79	120 - 79	119 - 79
0.75	117 - 79	108 - 79	106 - 79	104 - 79	104 - 79	104 - 79	103 - 79	103 - 79	103 - 79	103 - 79	103 - 79
1.00	108 - 79	101 - 79	99 - 79	97 - 79	97 - 79	96 - 79	96 - 79	96 - 79	96 - 79	96 - 79	96 - 79
1.25	104 - 77	97 - 77	94 - 77	93 - 77	92 - 77	92 - 77	92 - 77	91 - 77	91 - 77	91 - 77	91 - 77
1.50	102 - 76	94 - 76	91 - 76	90 - 76	89 - 76	89 - 76	88 - 76	88 - 76	88 - 76	88 - 76	88 - 76
1.75	99 - 63	92 - 75	89 - 75	87 - 75	87 - 75	86 - 75	86 - 75	86 - 75	86 - 75	86 - 75	86 - 75
2.0	98 - 59	90 - 70	87 - 70	86 - 70	84 - 70	83 - 70	82 - 70	82 - 70	81 - 70	81 - 70	81 - 70
2.5		87 - 63	83 - 63	79 - 63	77 - 63	75 - 63	74 - 63	74 - 63	73 - 63	73 - 63	72 - 63
3.0		77 - 48	78 - 57	73 - 57	71 - 57	69 - 57	68 - 57	68 - 57	67 - 57	67 - 57	66 - 57
3.5		74 - 45	74 - 53	69 - 53	67 - 53	65 - 53	64 - 53	63 - 53	62 - 53	62 - 53	62 - 53
4.0		71 - 42	71 - 50	66 - 50	63 - 50	61 - 50	60 - 50	59 - 50	59 - 50	58 - 50	58 - 50
5.0			58 - 37	61 - 44	58 - 44	56 - 44	55 - 44	54 - 44	53 - 44	53 - 44	52 - 44
6.0			55 - 34	50 - 34	55 - 40	52 - 40	51 - 40	50 - 40	49 - 40	48 - 40	48 - 40
7.0			53 - 31	48 - 31	52 - 37	50 - 37	48 - 37	47 - 37	46 - 37	45 - 37	45 - 37
8.0				46 - 29	43 - 29	48 - 35	46 - 35	44 - 35	43 - 35	43 - 35	42 - 35
9.0				45 - 28	42 - 28	39 - 28	44 - 33	43 - 33	42 - 33	41 - 33	40 - 33
10.0				44 - 26	40 - 26	38 - 26	42 - 31	41 - 31	40 - 31	39 - 31	38 - 31
11.0					39 - 25	37 - 25	35 - 25	40 - 30	39 - 30	38 - 30	37 - 30
12.0					38 - 24	36 - 24	34 - 24	33 - 24	37 - 28	36 - 28	36 - 28
13.0					38 - 23	35 - 23	33 - 23	32 - 23	36 - 27	35 - 27	35 - 27
14.0						34 - 22	33 - 22	31 - 22	30 - 22	34 - 26	34 - 26

Below Blue Line E_a = 2" - Above Blue Line E_a = 4"

0 to 5 MPH	5 to 10 MPH	10 to 15 MPH	15 to 20 MPH	20 to 25 MPH	25 to 30 MPH	30 to 35 MPH	35+ MPH

Figure 3.8: Freight Corridor Maximum Speed Improvement (MPH)

A = 15 feet, $E_{u,p,initial}$ = 3 inches, $E_{u,p,final}$ = 5 inches

Do	10	15	20	25	30	35	40	45	50	55	60
	Deflection Angle (I)										
0.50	182 - 79	170 - 79	166 - 79	164 - 79	162 - 79	162 - 79	161 - 79	161 - 79	161 - 79	161 - 79	160 - 79
0.75	158 - 79	144 - 79	140 - 79	138 - 79	137 - 79	136 - 79	136 - 79	136 - 79	135 - 79	135 - 79	135 - 79
1.00	145 - 79	132 - 79	128 - 79	125 - 79	124 - 79	123 - 79	123 - 79	123 - 79	122 - 79	122 - 79	122 - 79
1.25	138 - 77	124 - 77	119 - 77	117 - 77	116 - 77	115 - 77	115 - 77	114 - 77	114 - 77	114 - 77	114 - 77
1.50	133 - 76	119 - 76	114 - 76	111 - 76	110 - 76	109 - 76	109 - 76	108 - 76	108 - 76	108 - 76	108 - 76
1.75	129 - 63	115 - 75	109 - 75	107 - 75	106 - 75	105 - 75	104 - 75	104 - 75	104 - 75	103 - 75	103 - 75
2.0	98 - 59	111 - 70	106 - 70	103 - 70	102 - 70	100 - 70	99 - 70	98 - 70	98 - 70	97 - 70	97 - 70
2.5		102 - 53	100 - 63	95 - 63	92 - 63	90 - 63	89 - 63	88 - 63	88 - 63	87 - 63	87 - 63
3.0		77 - 48	94 - 57	88 - 57	85 - 57	83 - 57	82 - 57	81 - 57	81 - 57	80 - 57	80 - 57
3.5		74 - 45	89 - 53	83 - 53	80 - 53	78 - 53	77 - 53	76 - 53	75 - 53	75 - 53	74 - 53
4.0		71 - 42	78 - 42	80 - 50	76 - 50	74 - 50	72 - 50	71 - 50	71 - 50	70 - 50	70 - 50
5.0			73 - 37	74 - 44	70 - 44	68 - 44	66 - 44	65 - 44	64 - 44	63 - 44	63 - 44
6.0			55 - 34	63 - 34	66 - 40	63 - 40	61 - 40	60 - 40	59 - 40	58 - 40	58 - 40
7.0			53 - 31	60 - 31	63 - 37	60 - 37	58 - 37	56 - 37	55 - 37	54 - 37	54 - 37
8.0				58 - 29	54 - 29	57 - 35	55 - 35	53 - 35	52 - 35	52 - 35	51 - 35
9.0				45 - 28	52 - 28	49 - 28	53 - 33	51 - 33	50 - 33	49 - 33	48 - 33
10.0				44 - 26	51 - 26	48 - 26	51 - 31	49 - 31	48 - 31	47 - 31	46 - 31
11.0					49 - 25	46 - 25	44 - 25	48 - 30	46 - 30	45 - 30	44 - 30
12.0					48 - 24	45 - 24	43 - 24	41 - 24	45 - 28	44 - 28	43 - 28
13.0					38 - 23	44 - 23	42 - 23	40 - 23	44 - 27	42 - 27	42 - 27
14.0						43 - 22	41 - 22	39 - 22	38 - 22	41 - 26	40 - 26

Below Blue Line E_a = 2" - Above Blue Line E_a = 4"

0 to 5 MPH	5 to 10 MPH	10 to 15 MPH	15 to 20 MPH	**20 to 25 MPH**	**25 to 30 MPH**	**30 to 35 MPH**	**35+ MPH**

Figure 3.9: Freight Corridor Maximum Speed Improvement (MPH)

A = 15 feet, $E_{u,p,initial}$ = 3 inches, $E_{u,p,final}$ = 9 inches

Figures 3.10, 3.11, and 3.12 illustrate speed improvement potential for lines with predominantly passenger traffic. Both initial and final speeds are higher than the comparative freight corridor scenario due to curve superelevation designed for passenger instead of freight traffic. For an allowable curve shift A, the amount of speed improvement is higher for curves with a lower central angle value. For scenarios of the same degree of curve, the speed improvement reaches an asymptotic value at higher central angles. This characteristic is due to the fact that at low central angles, a realigned curve can feature a considerably shifted center point for the central circular curve. At high central angle values, the curve center point cannot be shifted to nearly the same extent without the realigned geometry escaping from the existing right-of-way. The passenger-traffic scenario can support substantially higher operating speeds than would be feasible using the same parameters in the freight-traffic scenario.

Deflection Angle (I)

Do	10	15	20	25	30	35	40	45	50	55	60
0.50	184 - 79	171 - 79	166 - 79	164 - 79	163 - 79	162 - 79	161 - 79	161 - 79	161 - 79	161 - 79	160 - 79
0.75	159 - 79	144 - 79	138 - 79	135 - 79	134 - 79	133 - 79	132 - 79	132 - 79	132 - 79	131 - 79	131 - 79
1.00	145 - 79	128 - 79	122 - 79	119 - 79	117 - 79	116 - 79	115 - 79	115 - 79	114 - 79	114 - 79	114 - 79
1.25	137 - 79	118 - 79	111 - 79	107 - 79	105 - 79	104 - 79	103 - 79	103 - 79	102 - 79	102 - 79	102 - 79
1.50	131 - 79	111 - 79	103 - 79	99 - 79	97 - 79	96 - 79	95 - 79	94 - 79	94 - 79	93 - 79	93 - 79
1.75	94 - 63	105 - 79	97 - 79	93 - 79	90 - 79	89 - 79	88 - 79	87 - 79	87 - 79	87 - 79	86 - 79
2.0	92 - 59	102 - 79	93 - 79	88 - 79	85 - 79	84 - 79	83 - 79	82 - 79	82 - 79	81 - 79	81 - 79
2.5		84 - 63	86 - 71	81 - 71	78 - 71	76 - 71	75 - 71	74 - 71	73 - 71	73 - 71	73 - 71
3.0		67 - 48	81 - 65	75 - 65	72 - 65	70 - 65	69 - 65	68 - 65	67 - 65	67 - 65	67 - 65
3.5		64 - 45	67 - 53	71 - 60	68 - 60	66 - 60	64 - 60	63 - 60	63 - 60	62 - 60	62 - 60
4.0		62 - 42	65 - 50	68 - 56	65 - 56	62 - 56	61 - 56	60 - 56	59 - 56	59 - 56	58 - 56
5.0			51 - 37	56 - 44	60 - 50	57 - 50	55 - 50	54 - 50	53 - 50	53 - 50	52 - 50
6.0			49 - 34	44 - 34	49 - 40	53 - 46	52 - 46	50 - 46	49 - 46	49 - 46	48 - 46
7.0			47 - 31	42 - 31	47 - 37	44 - 37	49 - 42	47 - 42	46 - 42	46 - 42	45 - 42
8.0				40 - 29	37 - 29	42 - 35	41 - 35	45 - 40	44 - 40	43 - 40	42 - 40
9.0				39 - 28	36 - 28	34 - 28	39 - 33	38 - 33	42 - 37	41 - 37	40 - 37
10.0				38 - 26	35 - 26	33 - 26	38 - 31	36 - 31	35 - 31	39 - 35	39 - 35
11.0					34 - 25	32 - 25	30 - 25	35 - 30	34 - 30	33 - 30	37 - 34
12.0					33 - 24	31 - 24	29 - 24	28 - 24	33 - 28	32 - 28	31 - 28
13.0					33 - 23	30 - 23	29 - 23	27 - 23	32 - 27	31 - 27	30 - 27
14.0						30 - 22	28 - 22	27 - 22	26 - 22	30 - 26	30 - 26

Below Blue Line E_a = 2", Above Blue Line and Below White Line E_a = 4", Above White Line E_a = 6"

0 to 5 MPH	5 to 10 MPH	10 to 15 MPH	15 to 20 MPH	20 to 25 MPH	25 to 30 MPH	30 to 35 MPH	35+ MPH

Figure 3.10: Passenger Corridor Maximum Speed Improvement (MPH)

A = 15 feet, $E_{u,p,initial} = E_{u,p,final}$ = 3 inches

Deflection Angle (I)

Do	10	15	20	25	30	35	40	45	50	55	60
0.50	198 - 79	186 - 79	182 - 79	180 - 79	179 – 79	178 - 79	178 - 79	178 - 79	178 - 79	177 - 79	177 - 79
0.75	170 - 79	156 - 79	151 - 79	149 - 79	147 – 79	146 - 79	146 - 79	145 - 79	145 - 79	145 - 79	145 - 79
1.00	145 - 79	138 - 79	133 - 79	130 - 79	128 – 79	127 - 79	127 - 79	126 - 79	126 - 79	126 - 79	126 - 79
1.25	137 - 79	127 - 79	120 - 79	117 - 79	116 – 79	114 - 79	114 - 79	113 - 79	113 - 79	113 - 79	112 - 79
1.50	131 - 79	119 - 79	111 - 79	108 - 79	106 – 79	105 - 79	104 - 79	104 - 79	103 - 79	103 - 79	103 - 79
1.75	107 - 63	112 - 79	105 - 79	101 - 79	99 – 79	98 - 79	97 - 79	96 - 79	96 - 79	96 - 79	95 - 79
2.0	92 - 59	102 - 79	100 - 79	96 - 79	93 – 79	92 - 79	91 - 79	90 - 79	90 - 79	90 - 79	89 - 79
2.5		91 - 63	92 - 71	88 - 71	85 - 71	83 - 71	82 - 71	81 - 71	81 - 71	81 - 71	80 - 71
3.0		77 - 48	87 - 65	82 - 65	79 - 65	77 - 65	76 - 65	75 - 65	74 - 65	74 - 65	74 - 65
3.5		74 - 45	74 - 53	78 - 60	74 - 60	72 - 60	71 - 60	70 - 60	69 - 60	69 - 60	68 - 60
4.0		71 - 42	71 - 50	74 - 56	71 - 56	69 - 56	67 - 56	66 - 56	65 - 56	65 - 56	64 - 56
5.0			58 - 37	61 - 44	66 - 50	63 - 50	61 - 50	60 - 50	59 - 50	58 - 50	58 - 50
6.0			55 - 34	50 - 34	55 - 40	59 - 46	57 - 46	56 - 46	55 - 46	54 - 46	53 - 46
7.0			53 - 31	48 - 31	52 - 37	50 - 37	54 - 42	52 - 42	51 - 42	50 - 42	50 - 42
8.0				46 - 29	43 - 29	48 - 35	46 - 35	50 - 40	49 - 40	48 - 40	47 - 40
9.0				45 - 28	42 - 28	39 - 28	44 - 33	43 - 33	46 - 37	45 - 37	45 - 37
10.0				44 - 26	40 - 26	38 - 26	42 - 31	41 - 31	40 - 31	44 - 35	43 - 35
11.0					39 - 25	37 - 25	35 - 25	40 - 30	39 - 30	38 - 30	41 - 34
12.0					38 - 24	36 - 24	34 - 24	33 - 24	37 - 28	36 - 28	36 - 28
13.0					38 - 23	35 - 23	33 - 23	32 - 23	36 - 27	35 - 27	35 - 27
14.0						34 - 22	33 - 22	31 - 22	30 - 22	34 - 26	34 - 26

Below Blue Line E_a = 2", Above Blue Line and Below White Line E_a = 4", Above White Line E_a = 6"

0 to 5 MPH	5 to 10 MPH	10 to 15 MPH	15 to 20 MPH	**20 to 25 MPH**	**25 to 30 MPH**	**30 to 35 MPH**	**35+ MPH**

Figure 3.11: Passenger Corridor Maximum Speed Improvement (MPH)

A = 15 feet, $E_{u,p,initial}$ = 3 inches, $E_{u,p,final}$ = 5 inches

Deflection Angle (I)

Do	10	15	20	25	30	35	40	45	50	55	60
0.50	220 - 79	218 - 79	213 - 79	211 - 79	209 - 79	209 - 79	208 - 79	208 - 79	207 - 79	207 - 79	207 - 79
0.75	186 - 79	182 - 79	176 - 79	174 - 79	172 - 79	171 - 79	170 - 79	170 - 79	170 - 79	169 - 79	169 - 79
1.00	158 - 79	162 - 79	155 - 79	152 - 79	150 - 79	149 - 79	148 - 79	148 - 79	147 - 79	147 - 79	147 - 79
1.25	147 - 75	148 - 79	141 - 79	137 - 79	135 - 79	134 - 79	133 - 79	132 - 79	132 - 79	132 - 79	131 - 79
1.50	131 - 79	139 - 79	130 - 79	126 - 79	124 - 79	123 - 79	122 - 79	121 - 79	121 - 79	120 - 79	120 - 79
1.75	107 - 63	123 - 75	122 - 79	118 - 79	116 - 79	114 - 79	113 - 79	113 - 79	112 - 79	112 - 79	111 - 79
2.0	92 - 59	118 - 70	117 - 79	112 - 79	109 - 79	108 - 79	106 - 79	106 - 79	105 - 79	105 - 79	104 - 79
2.5		102 - 53	108 - 71	102 - 71	99 - 71	97 - 71	96 - 71	95 - 71	95 - 71	94 - 71	94 - 71
3.0		77 - 48	94 - 57	96 - 65	92 - 65	90 - 65	89 - 65	88 - 65	87 - 65	86 - 65	86 - 65
3.5		74 - 45	89 - 53	91 - 60	87 - 60	84 - 60	83 - 60	82 - 60	81 - 60	80 - 60	80 - 60
4.0		71 - 42	78 - 42	87 - 56	83 - 56	80 - 56	78 - 56	77 - 56	76 - 56	76 - 56	75 - 56
5.0			73 - 37	74 - 44	77 - 50	74 - 50	72 - 50	70 - 50	69 - 50	68 - 50	68 - 50
6.0			55 - 34	63 - 34	66 - 40	69 - 46	67 - 46	65 - 46	64 - 46	63 - 46	62 - 46
7.0			53 - 31	60 - 31	63 - 37	60 - 37	63 - 42	61 - 42	60 - 42	59 - 42	58 - 42
8.0				58 - 29	54 - 29	57 - 35	55 - 35	58 - 40	57 - 40	56 - 40	55 - 40
9.0				45 - 28	52 - 28	49 - 28	53 - 33	51 - 33	54 - 37	53 - 37	52 - 37
10.0				44 - 26	51 - 26	48 - 26	51 - 31	49 - 31	48 - 31	51 - 35	50 - 35
11.0					49 - 25	46 - 25	44 - 25	48 - 30	46 - 30	45 - 30	48 - 34
12.0					48 - 24	45 - 24	43 - 24	41 - 24	45 - 28	44 - 28	43 - 28
13.0					38 - 23	44 - 23	42 - 23	40 - 23	44 - 27	42 - 27	42 - 27
14.0						43 - 22	41 - 22	39 - 22	38 - 22	41 - 26	40 - 26

Below Blue Line E_a = 2", Above Blue Line and Below White Line E_a = 4", Above White Line E_a = 6"

0 to 5 MPH	5 to 10 MPH	10 to 15 MPH	15 to 20 MPH	20 to 25 MPH	25 to 30 MPH	30 to 35 MPH	35+ MPH

Figure 3.12: Passenger Corridor Maximum Speed Improvement (MPH)

A = 15 feet, $E_{u,p,initial}$ = 3 inches, $E_{u,p,final}$ = 9 inches

3.6 Future Work

A methodology of broader scope would also consider the constraint of finite tangent distance between adjacent curves. For particularly sinuous routes, curve improvement projects may be limited by the encroachment of lengthened spirals on the tangent segments between curves and not by the speed benefit that could be achieved by any one curve in isolation. In addition, the proximity of curves to station platforms, bridges, and turnouts can further constrain feasible curve realignment projects. These constraints are outside the scope of this work but should be included in a broader methodology (along with the single curve maximum speed improvement presented here) for selecting an optimal set of running time improvements on a passenger rail corridor.

Future work might also consider more complex curve geometries including compound curves with connecting spirals as well as the complications added by a multiple track railway line. These geometries are likely to further increase the complexity of the solution process. Additional realignment scenarios that impact tangents between curves to reduce the total central angle might also be considered. This type of project might be especially useful at locations where there is limited tangent distance between reverse curves.

Alignment shifts that are constrained at other points along the curve and not just the mid-ordinate might also be analyzed. Instead of modifying offset equations for each individual curve, a separate process might be developed in which related existing obstacles anywhere on a curve could be converted to an equivalent obstacle at the mid-ordinate of the curve body. In this way the methodology presented here might be adapted for any number of constrained right-of-way scenarios.

Finally an evaluation could be made of existing spiral design criteria to determine whether or not contemporary practice results in over-conservative designs. In this proposed work, experience of high cant deficiency operations on railways both domestically and internationally, should be leveraged to maximize the benefit to cost ratio of domestic intercity passenger railway projects.

3.7 Conclusions

In this chapter I have outlined the magnitude of potential curve speed improvements for conventional passenger traffic for rail corridors with predominantly freight or passenger traffic. In this analysis, a number of general conclusions can be drawn about curve realignment projects.

1. Spiral design criteria varies considerably and has a major impact on maximum potential curving speed improvement
2. The AREMA spiral criteria may be overly conservative for high cant deficiency spiral design.
3. Speed improvement on low-central-angle curves of high degree is more sensitive to spiral criteria than degree of curve.
4. There is a higher potential speed improvement on curves of lower central angle for the same degree of curve and allowable track centerline shift.
5. Corridors with predominantly passenger traffic have higher speed improvement potential because superelevation is not constrained by the need to accommodate freight traffic requirements.

The results of this work will provide planners of passenger railway lines a better method to evaluate potential curving speed improvement benefits and some design parameters that should be considered when analyzing potential upgrade scenarios.

CHAPTER 4: A PROJECT SELECTION MODEL FOR IMPROVING RUNNING TIME ON PASSENGER RAILWAY LINES

4.1 Introduction

Recent proposals for expanded intercity passenger rail service in the United States have included plans for new dedicated high-speed lines as well as incremental improvements to existing Amtrak service. Improvements to existing services aim to accommodate faster and more frequent passenger train operation, generally on trackage owned and operated by heavy-axle-load freight railways. In recent years, numerous studies and reports have been commissioned by state agencies and the U.S. federal government to assess the feasibility of new or improved passenger rail services (Amtrak 2009a, 2009b, Franke and Hoffman 2007, Franke et al. 2008). The scope of improvements considered in each of these studies varies considerably depending on the existing route conditions and the proposed rail service changes. Feasibility studies are typically too broad in scope to consider improvement projects on specific route sub-segments and instead group a set of improvements together into two or three alternative scenarios based on maximum operating speed. More detailed reports may evaluate the performance benefits of specific projects in terms of capacity, reliability, or running time (Amtrak 2009a). The methodology for selecting the most cost effective improvement projects is not well defined and often relies on the judgment of experts involved in planning the corridor. In some cases, planning and resources have focused on achieving high maximum speeds for rail services (NYDOT 2012), when instead greater benefits to passengers could be achieved by increasing overall average speeds.

As was discussed in the introductory chapter, commercial passenger train schedules are composed of minimum running time and buffer time components. For a given train consist and route, there is a minimum achievable running time that assumes no delay from passengers, other rail traffic, or external factors. Passenger train schedules typically have a second element of slack time distributed in different parts of the schedule to accommodate an expected amount of delay. The amount of slack time can vary depending on the track configuration along the route, conflicting rail traffic, train performance characteristics, and expected passenger boarding and alighting times. Sogin et al. (2011) demonstrated the capacity impacts of passenger train speed differential on single and double track rail networks. Although it is important to consider route capacity impacts caused by higher-speed trains, the focus of this research is on investments that improve minimum run time and not those addressing the slack portion of the schedule that is most sensitive to rail capacity constraints and resulting train delays. For example, in order to support the operation of higher-speed passenger service, renewal of the track structure might allow an increase in track class and therefore reduce minimum running time on the route. At the same time, additional segments of double track might be added to mitigate the loss in free capacity taken up by operating faster passenger trains and consequently reduce the potential delay from conflicting rail traffic. This research develops an optimization framework for the former type of investments to achieve higher speeds. The latter types of investments in capacity are not considered here because they do not change the minimum achievable running time.

Figure 4.1 illustrates running times for a passenger train travelling one mile at speeds corresponding to the Federal Railroad Administration (FRA) track classes 1 through 9. For a one-mile segment of track, the greatest marginal benefit in running time can be achieved by upgrading slower, rather than higher speed segments. For example, upgrading a route segment from FRA track class 2 to 3 reduces running time by 1 minute per mile, whereas upgrading a segment from track class 5 to 6 saves only 7 seconds per mile. Acceleration and braking events between segments of different operating speeds can diminish the already marginal running time benefit provided by higher speed improvements. Given a railway line with a distribution of existing operating speeds and segment specific upgrade and maintenance costs, there exists a lowest cost set of infrastructure conditions to achieve a target running time.

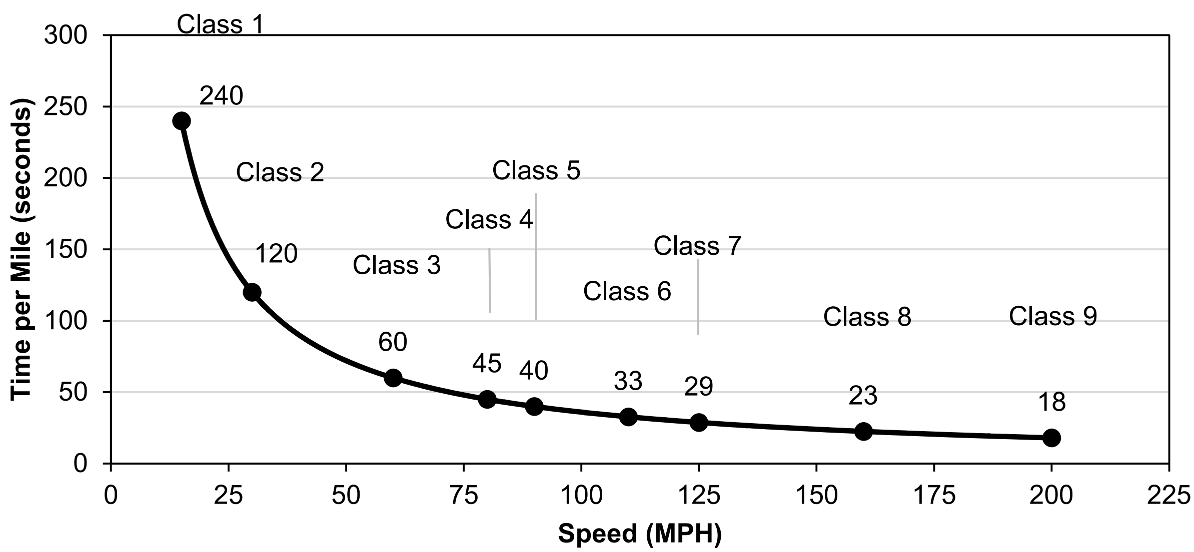


Figure 4.1: Running Time vs. Train Speed

Several alternatives exist for reducing running time and increasing average speeds, including improvements to track structure and geometry, signaling systems, highway grade crossings, and rolling stock (Wood and Robertson 2002). These types of projects cannot be evaluated in isolation because many of them offer different benefits depending on the condition of other components on the same or adjacent segments of the route. For example, consider a project to upgrade a single segment from a low maximum speed A to a higher maximum speed B. The cost of the upgrade from speed A to speed B is the same regardless of the condition of the adjacent segments. However, as illustrated in Figure 4.2, due to acceleration and braking effects, the incremental benefit of upgrading the intermediate segment will be greater for the case where the adjacent segments are already at the higher maximum speed B than it is for the case where the adjacent segments remain at the lower maximum speed. Thus, the benefit-to-cost ratio for the project to upgrade the intermediate segment varies greatly with the boundary conditions of adjacent segments. Given these complications and the multiple improvement options available, a formal methodology is needed to determine the relative cost effectiveness of upgrading different segments of a route. To meet this need, I developed an optimization model to be used as a decision support tool that rapidly and efficiently evaluates railway improvement strategies.

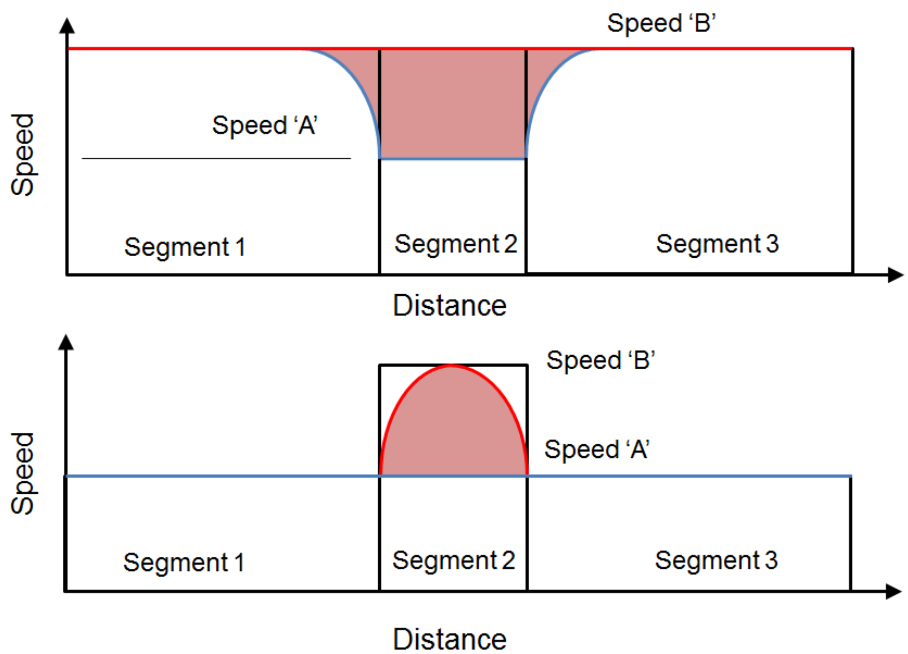


Figure 4.2: Change in Project Benefits Due to Speed Boundary Conditions

4.2 Literature Review

Several authors and research groups have investigated topics related to performance improvement of intercity passenger rail corridors. In the following section I review work ranging from feasibility studies of actual rail corridors to academic papers about rail and corridor planning.

Petersen and Taylor (2001) analyzed the economic feasibility of constructing a new railway line linking North and South Brazil. Traffic was assigned to different transportation modes and the optimal expansion path for constructing new railway links was determined. The optimization problem was formulated with nested dynamic programming models. This methodology might be modified to analyze the upgrade of existing transportation modes in addition to the construction of new segments.

Putallaz and Rivier (2004) presented a methodology that used a net present value (NPV) analysis for selecting capacity expansion, maintenance, and railway renewal projects. Their methodology uses three separate models, one that determines timetable alternatives, one that evaluates the stability of the alternatives, and one that evaluates the best maintenance and renewal strategies using the costs of track possession. When used together, the three models form a decision support framework for more efficiently planning train services and infrastructure on a railway network.

Martland (2006) discusses on-time performance of long distance passenger trains operating over freight railways. He states that the speed and reliability of rail service is largely a result of the rail infrastructure along the route. Several solutions for improving on time performance of rail services are identified, including adopting more achievable schedules, increasing operating discipline, and making investments in rail capacity. The author identifies experience-based schedules (with increased slack time) as the most promising alternative to increase passenger train on-time performance.

Beck et al. (2008) presented a NPV analysis that considers non-monetary benefits and lifecycle costs to compare the cost effectiveness of different signaling systems. Non-monetary benefits of different signaling systems were included through a weighted criteria assessment by industry experts. The ratio of qualitative benefit to lifecycle cost can be used as a decision support criteria when evaluating different signaling systems or methods of operation for a railway line.

Amtrak (2009a) published a report on the costs associated with incremental trip time improvements on the Northeast Corridor. This report was prepared to satisfy the requirements outlined in the Passenger Rail Investment and Improvement Act (PRIIA) of 2008. Amtrak outlines several projects along the corridor that have the potential of incrementally improving running times. These projects include both capacity expansion projects as well as projects that support higher speed operations. Running time benefits and costs for specific projects are outlined in the report appendix; however, the projects are not prioritized in the report in terms of their cost effectiveness.

Liu et al. (2010) presented a model that evaluates the benefits and costs of improving railway track segments for the purpose of derailment prevention. The authors calculated a net present value of track segments that considered the benefits resulting from reduced derailment risk and the costs associated with track class upgrades. The authors found that the monetary savings of derailment reduction alone did not outweigh the additional capital improvement and maintenance costs associated with higher track classes. While this work does not directly relate to passenger rail, it does present a methodology for consideration of track upgrades based on a present value analysis of segment-specific benefits and costs.

Perl (2012) discussed recent changes in the U.S. federal government policy towards intercity passenger rail. Following the presidential election of 2008, high speed and intercity passenger rail became a policy priority and new funding sources became available as a result of the American Reinvestment and Recovery Act (ARRA) High-Speed Intercity Passenger Rail

(HSIPR) program and Transportation Investment Generating Economic Recovery (TIGER) grants. These funds were allocated to dedicated, high-speed rail initiatives such as the California High Speed Rail project, as well as incremental improvements to existing routes, such as the *Lincoln Service* between Chicago, Illinois and St. Louis, Missouri.

Kanafani et al. (2012) evaluated the impact of speed on passenger rail service costs. This work focused on UIC International Railway Statistics for 2009 and considered national averages based on the proportion of high speed services. The results of this analysis confirmed that maintenance and energy costs of rail service increased with average operating speed. Labor costs, however, decreased with higher speeds. The study showed that the increased operating costs at higher speeds were outweighed by operational efficiencies, meaning that an optimal speed for HSR is the highest supported by revenue from passenger demand.

Lai and Po-Wen (2012) presented a framework for using mathematical programming to identify an optimal strategy to reduce running time on a passenger rail corridor. The problem is formulated as a mixed integer program that maximizes the reduction in travel time for different combinations of rolling stock and infrastructure. This model considered the performance of various trains given a set of line improvements, but the long length of the segments did not account for interaction effects between adjacent route segments. Potential improvements to this framework include considering the interaction effects of smaller project segments, as well as track maintenance costs over a time period.

Caughron et al. (2013) presented the first version of the project selection model illustrated in this chapter. This thesis chapter is directly adapted from this original paper which was presented at the American Railway Engineering and Maintenance of Way Association conference.

Tang et al. (2016) developed a refined version of the passenger rail project selection model. This work made the important refinement of incorporating train energy consumption and operating costs that greatly enhanced the utility of this type of model.

4.3 Project Selection Methodology

An ultimate process for planning improvements to a rail corridor is shown in Figure 4.3. In this process, existing or proposed rail service is evaluated for a given budget or service target. To meet this objective, planners may select from a range of potential corridor improvements that may include alternative improvements to infrastructure and/or rolling stock. Each of these improvements may have an impact on service quality metrics including frequency, running time, and reliability. Each of these three metrics has a potential impact on ridership and revenue depending on the passenger demand along the route. In addition to an impact on revenue, each improvement scenario has a characteristic initial capital and long term operating cost. The net present value (NPV). The improvement scenario with the highest NPV can be selected as the optimal strategy for the corridor.

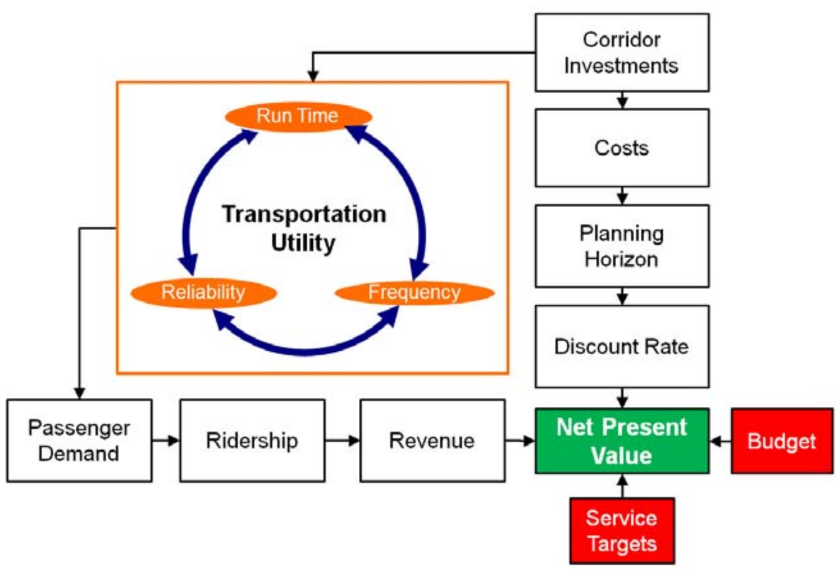


Figure 4.3: Ultimate Rail Service Optimization

This work focuses on a subset of these steps and considers infrastructure improvements that impact the minimum running time of a passenger service. The main objective is to define the relationship between running time and minimum present value capital and maintenance cost of infrastructure. Once this relationship has been established for a given route, the results can be applied to select an appropriate running time goal, infrastructure cost budget and suite of performance improvement projects. Future expansions of this work could incorporate frequency and reliability improvements into the model framework along with a ridership and revenue model to allow for a complete net present value analysis. When fully formulated in this manner, the resulting NPV could be used to define the initial budget through a feedback loop and the optimization framework run through a series of iterations until a stable equilibrium is reached. At this point, an optimal set of improvement projects that can be funded on the basis of future revenues is identified.

4.4 Project Selection Model Formulation

The proposed project selection model is formulated as a mixed integer program (MIP). Appendix Table A.1 shows the sets used in the model. A route is divided into segments that can be independently upgraded with each segment denoted by the index n. The index s is used to denote the different train speed states. The index $s*$ is an aliased index of s used for comparing different speed states. The index c is used to establish different infrastructure speed conditions.

The primary decision of the model $x_{n,c}$ is a binary variable with state 1 for track on segment n to be at condition c and 0 otherwise. Changes in the condition of this variable indicate that a project has been selected. The variable $v_{n,s,t}$ is the solution speed profile of all trains on the entire route. It is a binary variable that is 1 for train service t on segment n be at speed s and 0 for all other speed states for the same train and segment. Variable $z_{n,s,s^*,t}$ is a binary variable with state 1 when a train t changes speed from speed s to s^* between two adjacent segments. The variables $b_{n,t}$, which is the braking distance required from the previous to the current segment speed, and $a_{n,t}$, the acceleration distance from previous to the current segment speed, are used to constrain train speed profiles to those that are feasible for the route and train characteristics. At a high level, the model computes total train running time in a manner similar to a simple train performance calculator (TPC) while constraining the infrastructure condition to that which has a present value cost less than a specified budgetary amount.

$$\sum_{n=1}^{N}\sum_{s=0}^{S}\sum_{t=1}^{T}\theta_{n,t} l_n \delta_s v_{n,s,t} + \sum_{n=2}^{N}\sum_{s=0}^{S}\sum_{s^*=0}^{S}\sum_{t=1}^{T}\theta_{n,t}\tau_{s,s^*,t} z_{n,s,s^*,t} \tag{4.1}$$

$$\sum_{n=1}^{N}\sum_{c=0}^{C} p_{n,c} x_{n,c} \leq B \tag{4.2}$$

$$\sum_{s=0}^{S}\sigma_s v_{n,s,t} \leq \sum_{c=0}^{C} \nu_c\, x_{n,c} \tag{4.3}$$

$$z_{n,s,s^*,t} \leq v_{n,s^*,t} \quad \forall\, n, s, s^*, t \tag{4.4}$$

$$z_{n,s,s^*,t} \leq v_{n-1,s,t}\ \forall\, n, s, s^*, t \tag{4.5}$$

$$z_{n,s,s^*,t} + 1 \geq v_{n,s^*,t} + v_{n-1,s,t}\ \forall\, n, s, s^*, t \tag{4.6}$$

$$l_n - a_{n,t} - b_{n+1,t} \geq 0 \quad \forall\, n, t \tag{4.7}$$

$$b_{n,t} \geq \sum_{s=0}^{S}\left(\beta_{s,t} v_{n-1,s,t} - \beta_{s,t} v_{n,s,t}\right) \quad \forall\, 2 \leq n \leq N, t \tag{4.8}$$

$$a_{n,t} \geq \sum_{s=0}^{S} \left(\alpha_{s,t} v_{n,s,t} - \alpha_{s,t}\, v_{n-1,s,t} \right) \forall\, 2 \leq n \leq N, t \tag{4.9}$$

$$\sum_{s=0}^{S} \sigma_s v_{n,s,t} \leq h_{n,t} \quad \forall\, n, t \tag{4.10}$$

$$\sum_{s=0}^{S} v_{n,s,t} = 1 \quad \forall\, n, t \tag{4.11}$$

$$\sum_{c=0}^{C} x_{n,c} = 1 \quad \forall\, n \tag{4.12}$$

Equation 4.1 is the model objective function. The first summation computes the base minimum running time of all train services over the route given the length of the segment and selected speed state. The second summation adds an additional amount of time delay experienced by train services as they accelerate or decelerate between adjacent track segments of different speeds. It should be noted that this time component is not a delay in the traditional terminology of rail capacity, but rather a term that describes the additional time required by the rail service to change speeds over time versus instantaneously between segments. This acceleration and braking delay $\tau_{s,s^*,t}$ is pre-calculated outside the model using a TPC for the particular train consist associated with each train service. The total of these two objective function summations is the total running time of all services on the corridor. The train-service-importance factor $\theta_{n,t}$ represents the value of time for each type of train service on each segment. If the route features several rail service types with different performance characteristics, passenger ridership, and train frequencies, the importance factor can be adjusted to weight the running time of one train more than another. For example, consider a route where both commuter and intercity passenger services operate on the same trackage. By applying weighting factors of 0.25 and 1 to the commuter and intercity services respectively, the model will evaluate the running time of the intercity train as four times more important than the commuter train. By adjusting this parameter for different segments of the route, the impact of heavier ridership on certain segments of the corridor can be evaluated by the model. In this manner, the train-service-importance factor permits differences in revenue potential of different train services to be incorporated into the optimization procedure. The train-service-importance factor is not considered in the case study presented later in this work, which is focused only on establishing the cost versus running time relationship.

Equations 4.2 through 4.12 are model constraints. Equation 4.2 constrains the present value capital and maintenance cost of the route to less than a certain budget B. Equation 4.3 establishes that the train speed on a segment n cannot exceed the speed that is supported by the infrastructure condition. Equations 4.4 through 4.6 define the linking variable $z_{n,s,s*,t}$ which has the state 1 when there is a speed change between segments from state s to $s*$ and 0 otherwise. Figure 4.4 illustrates the condition of $z_{n,s,s*,t}$ with changes in the operating speed $v_{n,s,t}$ along several route segments. The correct amount of acceleration and braking time is added to the model objective function by summing the product of $z_{n,s,s*,t}$ and the acceleration and braking time parameter $\tau_{s,s^*,t}$ over all segments and trains.

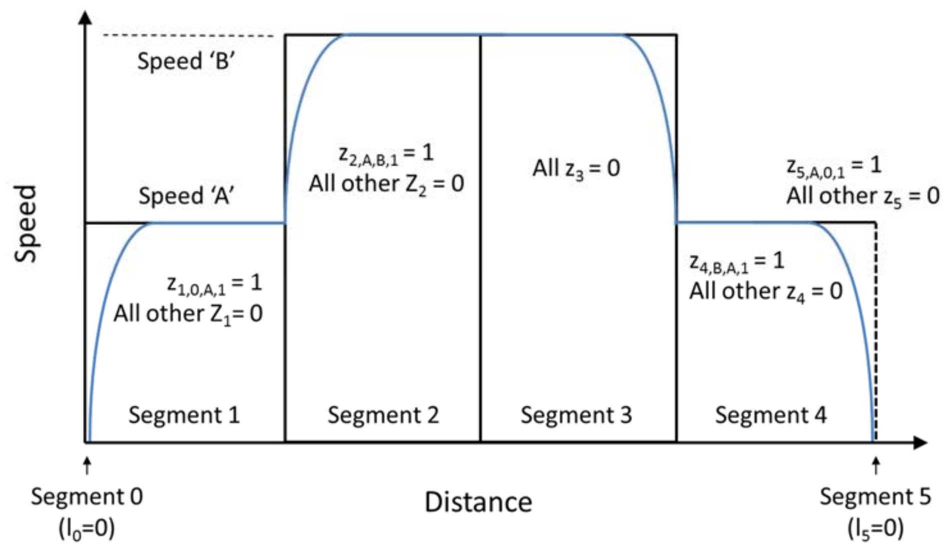


Figure 4.4: Illustration of the Function of $z_{n,s,s^*,t}$

Equations 4.7 through 4.9 compute the distance required for the train to accelerate and decelerate between different segments and ensure that the speed profile defined by $v_{n,s,t}$ is feasible for the given route conditions. Equation 4.7 ensures that there is sufficient distance to adequately change speeds within the length of a segment. This prevents the model from selecting an improved infrastructure condition with a speed so high on a segment that a train must begin braking for the next segment before it fully accelerates up to the maximum speed on that segment. Figure 4.5 illustrates the acceleration and braking distance constraints. The top half of the figure shows a train speed profile that satisfies these constraints, while the bottom half of the figure shows the distance constraints violated.

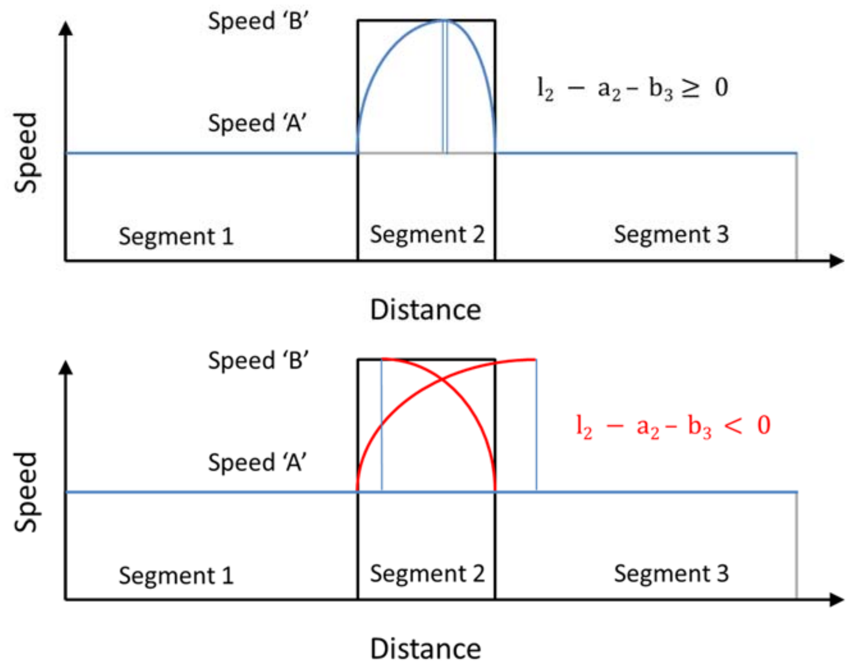


Figure 4.5: Illustration of Acceleration and Braking Distance Constraint

Equation 4.10 is used to establish a stopping pattern for different train services by setting $h_{n,t}$ to 0 for stations stops on segment n for train t. Equations 4.11 and 4.12 are constraints that ensure that there can only be one condition on each segment n for variables $v_{n,s,t}$ and $x_{n,t}$ respectively.

4.5 Case Study Scenario

A case study route was developed to demonstrate the functionality of the model. The route characteristics are based on a segment of a typical Midwest regional intercity passenger rail corridor. The portion of the rail corridor analyzed in this work is 48.1 miles long and features 15 curves, 74 highway grade crossings, and eight station locations. The maximum operating speed is 79 MPH, but there are several segments where speed is restricted to less than that due to curves or the lack of a signal system. These speed restrictions reduce the running time of trains on the corridor. The route was divided into 49 segments of average length 0.98 miles, with each segment having a distinct set of physical and operating characteristics (Figures 4.6 and 4.7).

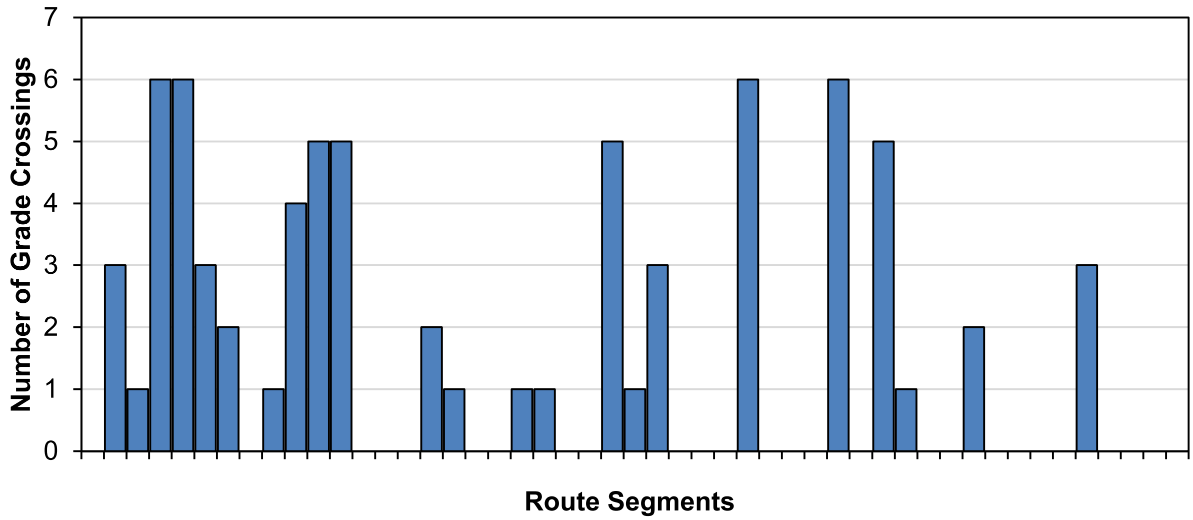


Figure 4.6: Grade Crossings on Route

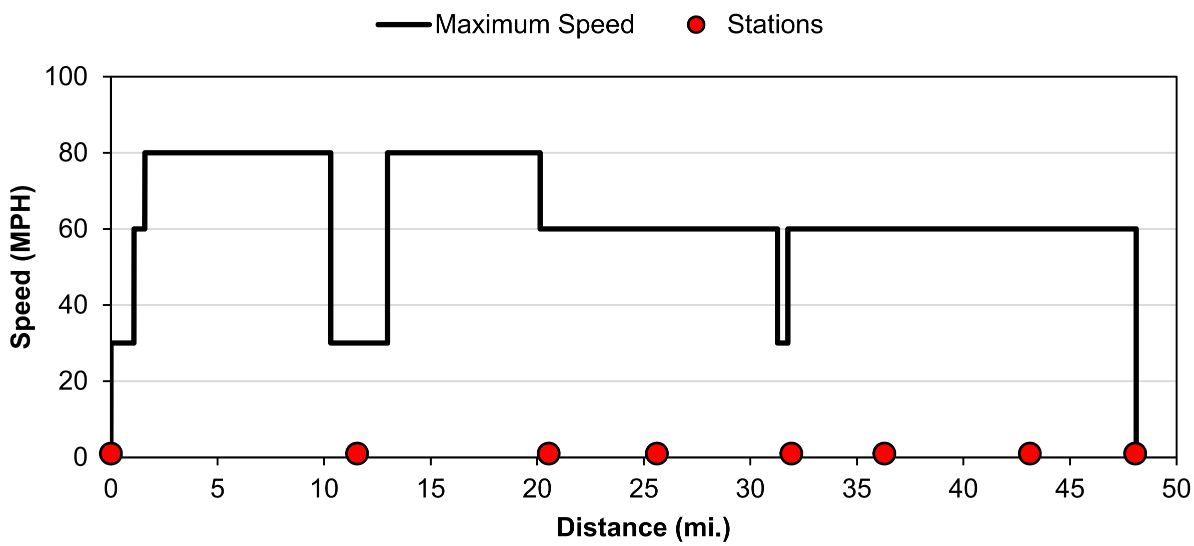


Figure 4.7: Route Speed and Location of Stations

Two train services were introduced on the case study route (Figure 4.8). An express service operates from endpoint to endpoint without stopping at any intermediate stations. A commuter service also operates from endpoint to endpoint but features six intermediate station stops.

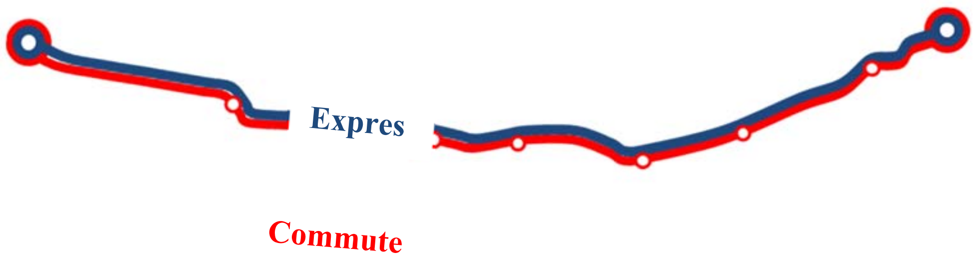


Figure 4.8: Route Map Illustrating the Rail Service Stopping Patterns

4.6 Case Study Input Data

The characteristics of the existing railway track and associated infrastructure have a major impact on capital improvement and maintenance costs. Infrastructure improvements were considered in the scope of potential projects if the improvement in question would help support higher train operating speeds. The differences in physical characteristics of each route segment means that each segment has a different relationship between capital cost, maintenance cost, and maximum operating speed. Four main elements were considered when estimating capital and maintenance costs on each segment: track structure, track geometry, signal system, and number of highway grade crossings.

Track structure refers to the system of components including rails, ties, fastening system, ballast, and subgrade. For an existing track, some of these components may need rehabilitation or replacement to support higher operating speeds. For example, increasing passenger train speeds from 30 to 60 MPH requires an increase in FRA track class from 2 to 3. Depending on the present segment condition, more higher quality cross ties in a given segment may be required in order to meet track safety regulations (FRA 2013b). In addition to these requirements, railway companies often have engineering policies that dictate use of premium track components for higher operating speeds, even if they are not required by federal regulations.

More sophisticated signaling and grade crossing warning systems are required for higher train operating speeds. The presence and condition of existing signaling equipment on a line has an effect on the incremental improvements and consequent costs of upgrading the corridor to support higher speeds. If a track segment in question is already equipped with cab signals or automatic train stop, the marginal cost of upgrading track speed from 79 to 90 MPH would be less than an equivalent segment that is not already so equipped. The cost of upgrading highway crossings to include features like four quadrant gates and intrusion detection to support higher passenger train speeds is also a major factor in segment-specific improvement costs. The requirements in each of these categories to support certain operating speeds are dictated by regulatory requirements and/or the engineering policies of the railway or agency in charge of the rail infrastructure.

The present-value cost for each segment is determined by the sum of the capital costs and present value maintenance costs over a time period. A spreadsheet calculator was developed to compute improvement costs specific to the characteristics of each segment. Capital costs for each segment were determined using the cost estimation methodology outlined by Quandel Consultants (2011) as a guideline. In order to improve operating speeds on a track segment, the infrastructure changes were considered when upgrading to a specific track class (Table 4.1). These input data were used solely to illustrate this case study and are not necessarily representative of the types of improvements necessary to support higher speeds on any specific corridor.

Table 4.1: Infrastructure Improvement Chart

Track Class	Max. Speed (MPH)	Track Structure	Signaling System	Grade crossings / miscellaneous
Class 3	60	Replace 1/3 Cross Ties (wood) , 136RE CWR, Surfacing		Curve shift
Class 4	79	Replace 1/3 Cross Ties (wood), 136RE CWR, Surfacing	CTC	Curve shift
Class 5	90	Replace 1/3 Cross Ties (wood), 136RE CWR, Surfacing	CTC / ATS or ATC	Curve shift, Four quad gate crossings
Class 6	110	Replace 2/3 Cross Ties (wood), 136RE CWR, Surfacing	CTC / ATS or ATC	Curve shift, four quad gate crossings with intrusion detection, fenced ROW

As an example, in order to improve a segment from FRA class 3 to class 5 standards, the segment would require one third of the cross-ties to be renewed, new 136RE continuous welded rail, ballast and surfacing, a centralized traffic control (CTC) system with automatic train stop (ATS) or automatic train control (ATC) system (if not already present), a potential shift in curve geometry (if required), and a four quadrant gate warning system at each highway grade crossing. For the case study route, all existing grade crossings were assumed to have flashing lights with conventional two-arm gates. The costs of a positive train control (PTC) system were not included in the analysis because this technology is to be implemented on all passenger routes regardless of speed.

Two potential curve geometry projects were considered for curve segments that limit train operating speed. In the first case, the curve superelevation and spiral lengths were increased to maximize the allowable train speed on the curve. This cost involves adjusting the track geometry within the footprint of the existing roadbed. The second project alternative involves shifting the curve so that its degree of curvature is minimized to the extent possible within the width of the right of way. In this case study, the right of way was assumed to be 100 feet wide, but this will differ on a location specific basis. For short length curves of low initial degree of curvature, a greater reduction in degree of curve is possible than for longer and higher degree curves. Unit costs for these two types of curve projects were also based on those used by Quandel Consultants (2011). For any curved segment, there is a maximum speed benefit possible through each type of improvements. In order to constrain the model to only select feasible speeds for each curved segment, a large cost penalty is added to the present value parameter for higher curve speed conditions not possible with the two improvement scenarios.

Annual steady state maintenance costs for the different track classes were based on those used by Zarembski and Resor (2004) who generated maintenance cost estimation tables for shared passenger and freight rail lines of different operating characteristics. For the purposes of this case study, the category of light-curvature, timber cross tie track at a traffic level less than 5 MGT per year was selected. For the purposes of this scenario, it is assumed that the combined annual tonnage of freight and passenger traffic would not exceed this threshold. The difference between Zarembski et al's "high" and "low" cost estimates was used as the incremental maintenance cost allocated to the passenger service operator. These costs were converted into a present-year value using a 10-year analysis period and a 5-percent discount rate. A limitation of the maintenance cost information used here is that we assumed the same maintenance cost for track classes 1-4. The model results would be further validated by increased accuracy in maintenance cost estimation for different track classes and features (e.g. grade crossings) specific to each segment.

Performance information on the rolling stock used in this case study is illustrated in Appendix Figures A.1 and A.2. The passenger train consist was based on typical Amtrak regional, intercity trainsets capable of 110 MPH operation and used on short-haul, regional corridors in the Midwestern United States. The consist has one 4,250 horsepower, four-axle locomotive, six single level passenger coaches, and an additional locomotive not providing tractive force but serving as a cab car and source of head-end power for passenger amenities. The rolling stock characteristics are summarized in Appendix Table A.4 Acceleration and braking time-distance calculations were performed using a simplified train performance calculator with one-second speed calculation steps. Acceleration distances and times were computed using the Canadian National train resistance formulas and coefficients as shown in chapter 16.2.1 of the AREMA manual for railway engineering (AREMA 2010a). Braking distances and times were computed by assuming a constant braking force throughout the range of speeds. These assumptions of passenger train performance calculation were verified by comparing the results to actual time-distance data obtained from an Amtrak track geometry car operating on regularly scheduled passenger trains. Using all this information, a combined acceleration and braking delay table was computed for use in the model (Table A.5). Two additional assumptions are that the passenger train is a “point mass” that can immediately begin to accelerate upon encountering a higher speed limit and second that the grades on the route are level or not significant enough to greatly change the acceleration and braking characteristics of the passenger train. These assumptions were made to simplify the initial model formulation but as discussed in the Future Work section, future development of the model will include grade and train length effects.

4.7 Case Study Results

The mixed integer program and case study data were fed into Paragon Decision Technology AIMMS optimization software installed on a desktop computer with 16 GB of RAM and a 3.4 GHz quad core processor. The case study route had 26,999 decision variables and 75,902 constraints. The GUROBI 5.0 solver included with the AIMMS software was the most efficient for this problem type and was generally able to converge to an optimal solution in 1-2 minutes. Larger routes with more segments, infrastructure speed conditions, or train speeds would require more decision variables and therefore longer solution times. The model was solved for 16 separate scenarios with different budget constraints. Figures 4.9 and 4.10 reflect a representative $45M budget scenario, while Figures 4.11 and 4.12 summarize the results from all scenarios.

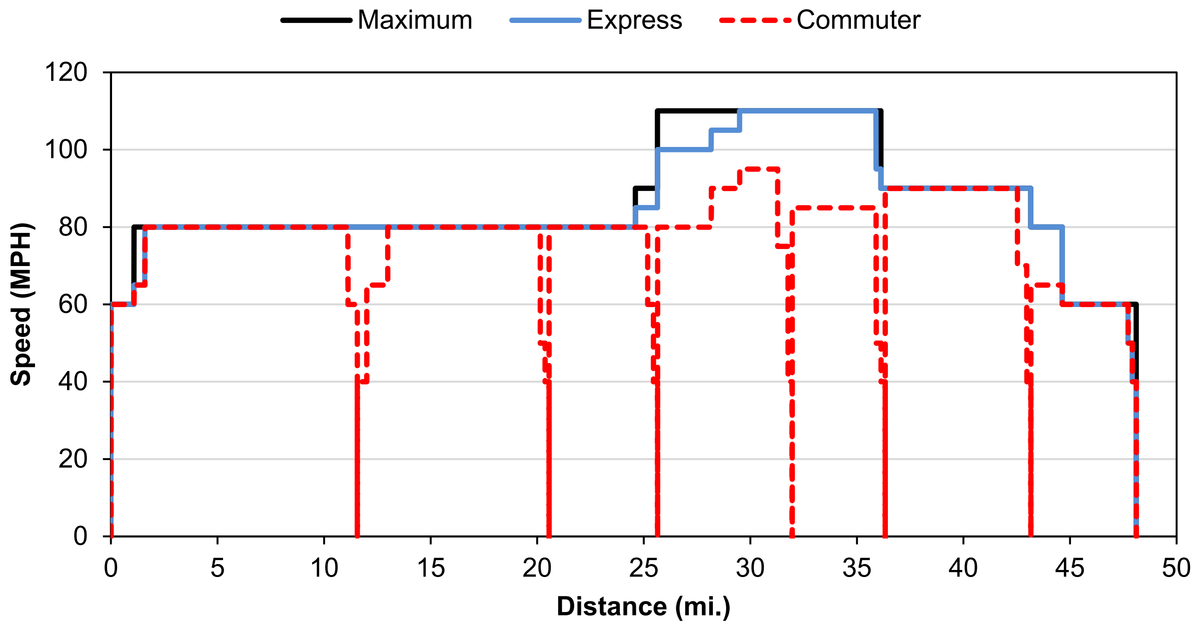


Figure 4.9: Optimal Route Configuration with $45M Present-Value Budget

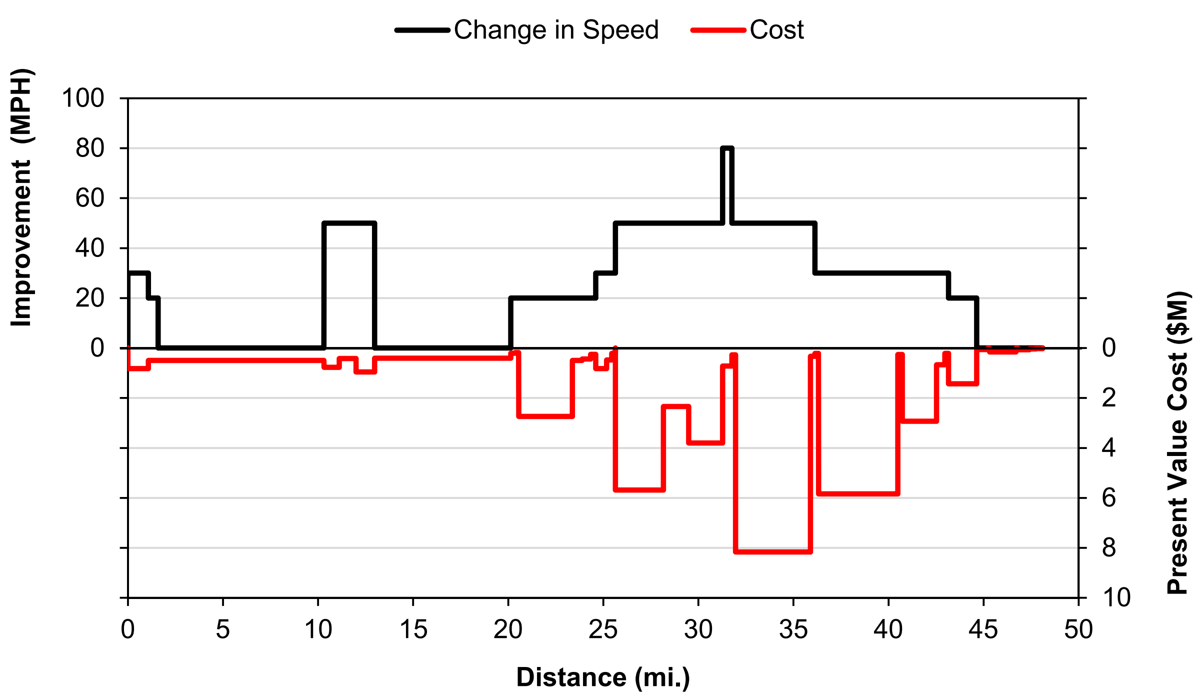


Figure 4.10: Speed Improvement vs. Cost with $45M Present-Value Budget

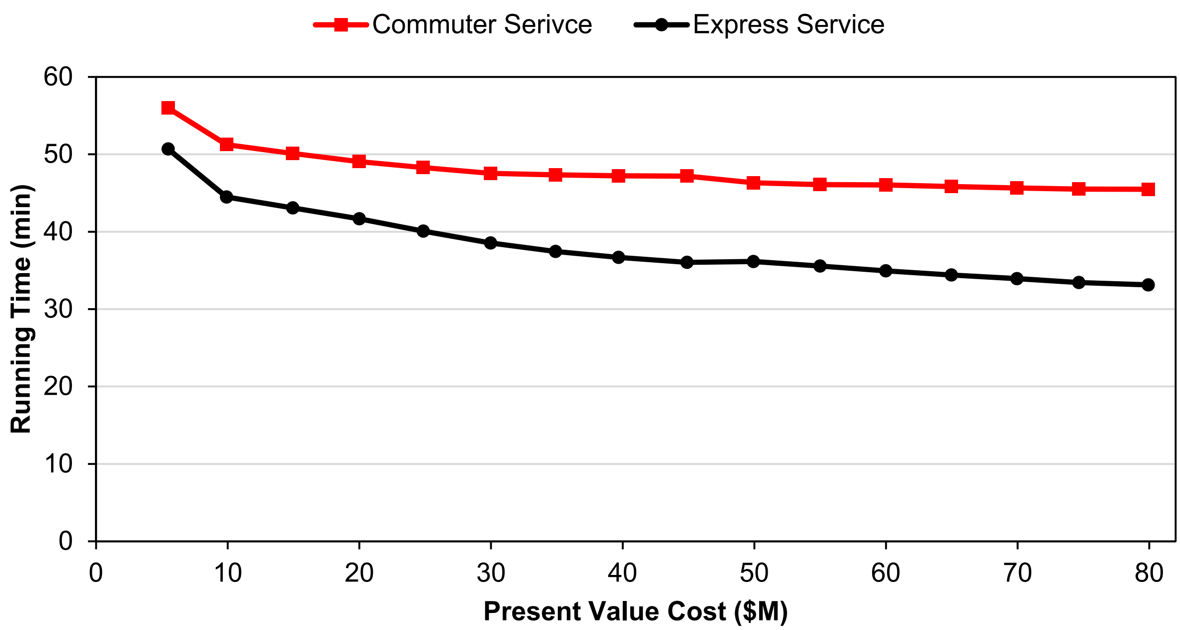


Figure 4.11: Running Time vs. Cost Relationship for Case Study Route

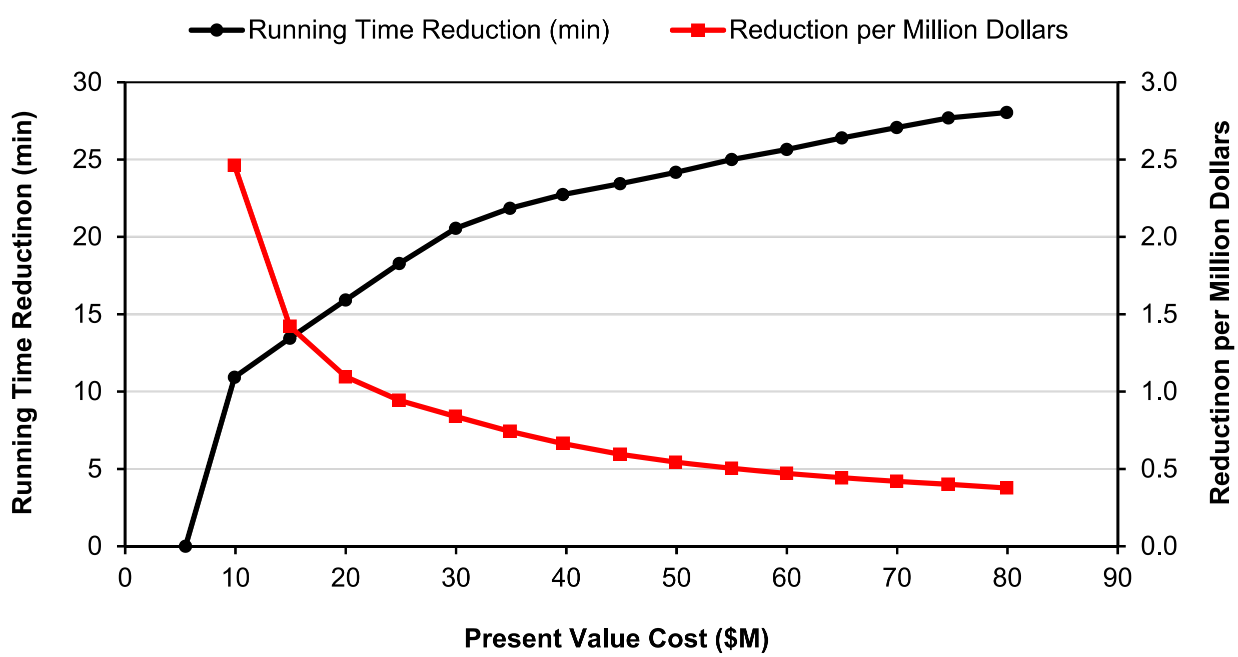


Figure 4.12: Running Time Reduction vs. Present Value Cost

Figure 4.9 shows the $45M optimal route configuration along with the performance of both train services along the route. The express service shown with the blue line generally meets the maximum track speed as shown in black. The commuter service, shown by the dashed red line, does not meet the maximum track speed on some segments due to train acceleration and braking characteristics as well as the short distance between intermediate stops along the route. It is clear that the segment improved to 110 MPH between mile 25 and 37 benefits the express service more so than the commuter service (Figure 4.9).

Figure 4.10 shows the speed improvement from the base configuration to the optimal configuration at a $45M present-value budget. The speed improvements are clustered in three distinct areas as shown by the black line. Due to the time delay incurred by braking and accelerating, the optimal pattern of upgrades for any budget case tends to exist in blocks rather than many separate individual segments. The red line in the lower portion of the figure reflects the present value cost of each segment along the route. Curves that had restricted the speed of the train around mile 10-13 and 32 are selected to be re-aligned for higher speeds by the model. In addition to the curve realignment, a segment of 110 MPH infrastructure is selected. On this segment, 21 grade crossings would receive four-quadrant gates and vehicle intrusion detection. The 110 MPH segment is bracketed by two segments of 90 MPH infrastructure. On these segments, 8 grade crossings would be upgraded to four-quadrant gates.

The relationship between the running time of the two passenger services and present value cost is shown in Figure 4.11. The red line shows the running time of the commuter service while the black line shows the running time of the express service at different scenarios. As the budget constraint was increased for each model scenario, there was a corresponding decrease in the running time of both services. In lower budget scenarios, the model solution can include improvements with only the highest benefit to cost ratio. As the budget is increased, the optimal solution includes further sets of improvements that are less cost effective in reducing running time and have a lower return on investment. The running time improvement of the commuter service is more limited by the acceleration and braking performance of the rolling stock at higher budget scenarios. In these scenarios there is a greater improvement in the running time of the express service because it can take advantage of relatively longer stretches of uninterrupted, higher speed running.

Figure 4.12 illustrates total running time reduction (black line) of both express and commuter services versus the present-value cost for the infrastructure configuration. The red line reflects the marginal benefit at each level of investment in minutes of running time reduction per additional million present-value dollars. As the level of investment in the existing corridor is increased, the marginal benefit decreases.

These figures and analysis techniques can be used to plan a series of infrastructure improvements for an intercity corridor. For an existing corridor, appropriate cost and performance data can be collected and input into the model to determine which segments are most cost effective to upgrade. Any proposed run time improvements to the corridor can be compared to a route and train specific frontier of optimal running time improvements (Figure 4.12). Proposed corridor configurations that are plotted far below the optimal cost versus running time reduction frontier should be re-evaluated against other alternatives. Different rolling stock alternatives could also be evaluated using the model framework by considering different train performance characteristics and comparing the resulting cost versus running-time curves shifted by present-value equipment costs. The rolling stock alternative that offers the lowest combined infrastructure and equipment cost for a given running-time reduction should be selected for use on the corridor.

4.8 Future Work

The framework presented in this chapter could be expanded to investigate a number of additional questions related to passenger rail corridors. More complex case studies might consider a more complete range of infrastructure improvements, including track classes 7 through 9 and the infrastructure capital and maintenance costs associated with electric traction. In addition, a more comprehensive analysis of the upgrade of curves could be performed. With more sophisticated capital cost estimation for improving curves, the cost effectiveness of curve realignment projects could be weighed against alternatives that substitute rolling stock with improved curving performance (e.g. tilting trains).

Although this chapter presents the model as a tool for analyzing intercity corridors, the same optimization techniques could also be applied to other rail corridor upgrade problems. Other forms of passenger rail transportation, such as light rail or metro systems might benefit from this type of optimization methodology in the planning of new lines or when evaluating alternatives for improving the performance of existing lines.

The model framework might also be adapted to look at freight rail corridors. Certain types of freight traffic are more time sensitive than others and might benefit from this optimization technique along the length of a corridor. In this freight scenario, the condition of the existing infrastructure could be rationalized in order to meet a certain travel time goal. It might be that on a given corridor, eliminating certain speed restrictions is more cost effective than maintaining track segments for 60 MPH or greater freight operation. The two most significant enhancements needed to adapt the model for this purpose would be to develop a more sophisticated approach to the effects of grade and train length on speed performance.

Finally, a more comprehensive corridor optimization framework might consider differences in operating costs for different train speed profiles. Fuel and equipment costs will be greater for higher operating speeds leading to an additional cost penalty for selecting higher versus lower speed improvements.

4.9 Conclusions

In this chapter I present an optimization model that can be used to help plan cost effective infrastructure improvements to improve running time performance of intercity passenger rail corridors. A mixed integer program is proposed that takes into account infrastructure capital costs, present-value maintenance costs, and the performance characteristics of the rolling stock operating on a route. By solving the model for a series of different budget constraints, a cost-versus-running-time function can be established. Using this relationship, potential improvements to the corridor can be evaluated against the optimal frontier of configurations generated by the model. With suitable cost data, this model could be incorporated into passenger rail corridor planning methodology and used as a decision support tool for passenger rail service planners.

CHAPTER 5: FUTURE WORK

In this thesis I focused on several aspects intended to improve the state of practice in planning infrastructure improvements to intercity passenger rail corridors. In Chapter 1 I presented an overview of intercity passenger rail planning in the United States. Chapter 2 described a Train Performance Calculator based in Microsoft Excel. Chapter 3 outlined relationships between curve geometry parameters and speed improvement potential. Finally, in Chapter 4 I presented an optimization model for selecting running time improvement projects for a passenger rail corridor. Transportation planners and engineers can incorporate the methods presented in this thesis as part of the planning process for passenger rail corridors. Although many of the concepts presented in this work can be readily applied, there are numerous refinements that could be made to further enhance the accuracy and utility of these planning tools. An outline of potential refinements is presented in this section, along with examples of future work and application of concepts to rail transportation planning problems.

5.1 Project Selection Model Incorporating Capacity and Reliability Improvements

One of the most important enhancements to the project selection methodology would allow planners to simultaneously optimize running time improvements with capacity and reliability improvements. As an example, extended sidings or added segments of second main track to a predominantly single-track route might allow passenger train schedules to be shortened. In this case, the minimum running time over the route would not change; however, the amount of schedule slack time previously needed could be reduced. There are many passenger rail corridors where it is probably more cost effective to reduce buffers that are built into schedules to accommodate delays, rather than trying to increase speeds to reduce running time. Put another way, rather than expending funds to hurry up, only to wait for other traffic, it may be more cost effective to reduce the need to wait. In addition to reducing buffer times, capacity upgrades could enable increased passenger train frequency. Such an increase in service frequency might increase ridership thereby generating more revenue. Expanding the number of trains on a route would allow infrastructure costs to be spread over a greater number of passengers.

Incorporating capacity and reliability improvements would require a reformulation of the model presented in Chapter 4. An alternate objective function would be required that incorporates the benefits resulting from both capacity and running time reduction projects. To evaluate running time reduction, capacity, and reliability in the same objective function, each parameter must be transformed to the same unit of measure. Passenger utility or alternatively service revenue could serve as common units for these parameters.

Decision variables governing capacity and reliability improvements could be established for each segment of the route, with a new set of constraints established to govern extending double track and sidings while simultaneously considering operating speed. This may result in a optimization model that is too difficult to solve. Alternatively, a pre-determined set of capacity and reliability projects could be entered in the model and evaluated by an alternate set of decision variables not linked to the route characteristics as in present model formulation. The benefits, costs, and any interdependencies of projects would be entered into the model as parameters and constraints. In this scenario, running time would be weighted by the amount of passenger utility or revenue expected with each incremental improvement. This objective function would then be maximized subject to the set of budget constraints used for the model case study.

5.2 Project Selection Model Incorporating Train Operating Costs

The project selection model as presented in Chapter 4 evaluates the capital improvement and maintenance costs of different infrastructure configurations. The cost component of the model could be refined further by incorporating train operating costs into the optimization process. Labor costs, energy or fuel costs, and rolling stock maintenance costs could be added to the model formulation. Energy costs will likely vary considerably depending on the type of rolling stock in service and the speed profile of the route and service schedule. A schedule with frequent stops or a route with frequent changes in speed will likely have higher energy costs than an otherwise similar route or schedule with more uniform speeds. In addition, the number of locomotives added to a train will affect both acceleration performance and energy costs of the service. Different strategies for locomotives could be evaluated using the model framework in order to determine the relative cost effectiveness of added locomotives for different routes and stopping patterns compared to other methods of running time reduction.

Maintenance costs will vary based on the type of rolling stock in service on the corridor. Labor costs will also vary depending on running time, but will also depend on the labor agreement in place for a particular route. Even so, significant reductions in the running time of the service may allow for the same number of passenger services to be operated with fewer trainsets and crews. This concept is analogous to train velocity in the freight rail sector. With a modest additional effort, these operating costs could be incorporated into the project selection model and likely with the same decision variables that are already used.

The analysis outlined in Chapter 3 illustrated the great potential for improving curving speed for passenger trains on many conventional routes. For routes that have a high proportion of speed restricting curves, investments that increase curving speed may be more cost effective than those that increase maximum operating speed. For an existing route, changes to curve geometry may be possible and would serve to increase curving speed by, reducing degree of curvature, increasing superelevation, and lengthening spirals. These improvements may not always be practical or cost effective due to a number of constraining factors, including freight rail traffic and right-of-way constraints. For some routes, tilting equipment may offer substantial running time benefits over conventional rolling stock for a relatively modest investment. This equipment investment strategy should be evaluated in the same project selection process and infrastructure running time improvements. In this case the model can be solved several times using a set of rolling stock types and budget constraints. Plotting the set of solutions creates a series of curves showing the optimal relationship between running time, budget, and rolling stock type. Before final comparison, the curves should be shifted by the marginal cost of the rolling stock type represented in the curve . It may be that for many routes the most effective investment strategy is to invest in tilting equipment before pursuing infrastructure upgrades to increase maximum operating speed.

5.3 Applying the Project Selection Model to Transit and Freight Rail Problems

The project selection methodology could be applied to other rail transportation problems, including urban rail transit and freight rail sectors. Although this thesis is focused on the application of the model to intercity passenger rail corridors, it can also be used for rail transit project selection. There are numerous urban rail transportation systems that for historical reasons have segments that restrict the operating speed of passenger trains. Examples of these segments include sharp curvature or segments at grade where rail vehicles must comingle with motor vehicle traffic. Due to right-of-way constraints that are inherent to urban rail systems, improvement projects that serve to increase speed through these constraints are costly. Potential improvement projects to address these slower segments might include curve realignments, segments of elevated trackage, or segments of subway trackage. In addition to running time improvements, different rolling stock types with characteristic acceleration and braking rates might be evaluated. Due to the dense nature of stations stops on many urban passenger rail systems, more expensive rolling stock with better acceleration and braking performance might be a more cost effective way of reducing running time when compared with infrastructure running time improvements.

A derivation of the project selection model might also be used for planning running time improvements to freight corridors. Intermodal, automotive, and perishable freight traffic can be more time sensitive than other commodities and therefore demand rail transit times comparable to those provided by trucking companies. Rail capacity and train delay are very important factors that influence transit time and are particularly important on high density, high speed freight corridors. On the other hand, improvements that focus on increasing freight train speeds and reducing minimum running time may in some cases be more cost effective than improvements that increase capacity. One example of this case might be a secondary main line with lower traffic volume and track maintained to FRA class 1 and 2 standards. In this scenario, infrastructure managers might be interested on what segments would be most cost effective to upgrade to FRA class 3 standards in order to achieve a certain running time reduction. Existing route conditions, including the condition of major bridges, track geometry, grade crossings, signaling systems, and other factors would influence the improvement cost of different segments. These segment by segment cost variances could be evaluated in much the same manner as in the intercity passenger rail case study presented in Chapter 4.

There are several key obstacles that must first be overcome prior to using the project selection model for freight corridor planning. By their nature, efficient freight trains are quite long and can span two or more route segments simultaneously. The manner in which the project selection model is currently structured does not consider the length of the train when computing the running time of different infrastructure configurations. This shortcoming might be overcome with a train specific route segmentation in a manner similar to what is described in Chapter 2 for the Excel based TPC. A more difficult obstacle to overcome is the impact of grades and curves on freight train acceleration and braking. Due to the inherently low power to weight ratio of modern North American freight trains, the impact of route grades and curvature is a key factor that should be evaluated in the project selection methodology. The acceleration and braking time parameter might be expanded with a route segment index. The running time would then be computed with a train specific acceleration and braking time that would vary segment to segment. At a minimum, extensive TPC testing of hypothetical speeds on a given route would need to be undertaken to populate this time parameter prior to using the model. For some cases it may be just as effective, and faster, to plan a freight corridor using a TPC to evaluate running times over a smaller, defined set of infrastructure improvement alternatives.

5.4 Network Application of Project Selection Model

In addition to supporting a strategic planning process for one route, the methodology presented in this paper might be used to optimize investment in a network of routes. The first step for this method involves solving the model several times for a given route with a set of budget constraints. Plotting the set of resulting solutions creates a curve showing the optimal relationship between running time and budget. This characteristic curve is generated for each route that is part of the network. The running time reduction for each route is next normalized by the relative differences in passenger utility. As an example, a five-minute reduction in running time on a route with only two daily passenger trains in most cases would offer less total benefit than an otherwise identical five-minute reduction on a route with twenty daily passenger trains. After the running time reductions are normalized, an optimal investment strategy can be determined on a network level. Although this method might be useful for network investment planners, in practice the political nature of passenger rail in the U.S. may tend to confound an otherwise rational and objective investment strategy.

5.5 Closing Comments

Passenger rail is expected to play an important role in an integrated, multi-modal transportation network for the foreseeable future. Use of the methods presented in this thesis can advance the state of the art of passenger rail corridor planning to maximize the benefit of both public and private investment in passenger rail transportation. By leveraging operations research, simulation models, and decision support tools, the planners of passenger rail corridors can more scientifically plan investments and operating strategies for corridors, increasing the effectiveness of passenger rail and bolstering public confidence in this important and fascinating transportation mode.

REFERENCES

Alpert, D, Kenton, M. 2011. *The Evolution of Amtrak, 1971-2011*. Greater Greater Washington. Online publication. URL: https://ggwash.org/view/9187/the-evolution-of-amtrak-1971-2011. Accessed on 12/11/2017.

American Railway Engineering and Maintenance-of-Way Association (AREMA). 2010a. *2010 Manual for Railway Engineering. Vol. 4. Chapter 16.2*. American Railway Engineering and Maintenance-of-Way Association, Lanham, MD.

American Railway Engineering and Maintenance-of-Way Association (AREMA). 2010b. *2010 Manual for Railway Engineering. Vol. 1. Chapter 5*. American Railway Engineering and Maintenance-of-Way Association, Lanham, MD.

Andersson, E, Bahr, H.V., and Nilstam, N.G. 1995. Allowing Higher Speeds on Existing Tracks – Design Considerations of the X2000 Train for Swedish State Railways. *Proceedings of the Institution of Mechanical Engineers, Part F: Journal of Rail and Rapid Transit* 209: 93-104.

Andersson, E., Nilstam, N.G., and Ohlsson, L. 1996. Lateral Track Forces at High Speed Curving, Comparisons of Practical and Theoretical Results of Swedish High Speed Train X2000, *Vehicle System Dynamics: International Journal of Vehicle Mechanics and Mobility* 25:S1, 37-52.

Avery, R. M. 1985. A coordinated visual representation of train performance, power, and energy consumption. Journal of *IEEE Transactions on Industry Applications* 2: 291-294.

Beck, K., Scheier, B., and Jaeger, B. 2008. Determining Investment Strategies for Railway Signaling through Simulation. *WIT Transactions on the Built Environment* 103: 65-74.

BNSF Railway (BNSF). 2010. Engineering Instructions. BNSF Railway, Fort Worth, TX.

Caughron, B.M., Dick, C.T., and Barkan, C.P.L. 2013. A Project Selection Model for Improving Running Time on Passenger Rail Corridors. *Proceedings, American Railway Engineering and Maintenance of Way Association Annual Conference, Indianapolis, IN,* 2013.

CN Engineering. 2002. CN Recommended Method RM 1305-0, Method for Determining Superelevation, Spiral Length, and Maximum Train Speed on Curves. Canadian National Railway Company, Montreal, QE.

Easa, S.M. 1991. Practical Optimization Model for Realigning Railway Horizontal Curves. *Journal of Civil Engineering Systems* 8: 27-36.

Federal Railroad Administration (FRA). 2013a. Vehicle/Track Interaction Safety Standards; High-Speed and High Cant Deficiency Operations. *Federal Register* 78: 16052-16126.

Federal Railroad Administration (FRA). 2013b. Code of Federal Regulations, Title 49 Volume 4: Part 213 - Track Safety Standards. Federal Railroad Administration, Washington, D.C.

Franke, M.W., and Hoffman, R.P. 2007. *Feasibility Report on Proposed Amtrak Service: Chicago-Rockford-Galena-Dubuque*. Illinois Department of Transportation, Springfield, IL. URL: http://www.dot.il.gov/amtrak/RCK_Feasibility.pdf. Accessed on 3/18/2013.

Franke, M.W., Hoffman, R.P., and Hilblom, B.E. 2008. *Feasibility Report on Proposed Amtrak Service from Chicago, to Iowa City, Iowa via Quad Cities.* Illinois Department of Transportation, Springfield, IL. URL: http://www.iowadot.gov/iowarail/passenger/grantapplication.htm. Accessed on 3/18/2013.

Harris, H.R., Schmid, E., and Smith, R.A. 1998. Introduction: Theory of Tilting Train Behavior. *Proceedings of the Institution of Mechanical Engineers*, *Part F: Journal of Rail and Rapid Transit* 212: 1-5.

Hay, W.W. 1982. *Railroad Engineering*, 2nd ed. John Wiley & Sons, Inc, New York City, NY.

Kanafani, A., Wang, R., and Griffin, A. 2012. The Economics of Speed – Assessing the Performance of High Speed Rail in Intermodal Transportation. *Procedia – Social and Behavioral Sciences* 43: 692-708.

Kerchof, B. 2012. Effect of Superelevation on Vertical and Lateral Wheel Forces. *Railway Track and Structures,* November 2012. American Railway Engineering and Maintenance of Way Association, Lanham, MD.

Klauser, P. 2005. Operating at High Cant Deficiency. *The Journal of Wheel/Rail Interaction*, October, 2012. Online publication. URL: http://www.interfacejournal.com/features/09-05/cant/1.html. Accessed on 3/13/2013.

Kolig, H, and Hesser, K.R., 1997. Tilt System for Amtrak High Speed Trainsets. In: *Proceedings of the 1997 IEEE/ASME Joint Rail Conference*, Boston, MA, March 1997, pp.59-64.

Krueger, H., Vaillancourt, E., Drummie, A. M., Vucko, S. J., and Bekavac, J. 2000. Simulation within the Railroad Environment. In: *Proceedings of the 32nd conference on Winter simulation,* Orlando, FL, December 2000 pp. 1191-1200.

Kufver, B. 2005. *Tracks for Tilting Trains*. European Commission Competitive and Sustainable Growth Programme–Fast And Comfortable Trains*,* Report D 8.

Lai, Y-C, and Po-Wen, H. 2012. High Speed Route Improvement Optimizer. *Transportation Research Record: Journal of the Transportation Research Boar*d 2289: 18-23.

Liu, X., Saat, M.R., Barkan, C.P.L. (2010). Benefit-Cost Analysis of Infrastructure Improvement for Derailment Prevention. In: *Proceedings of the 2010 Joint Rail Conference*, Urbana, IL, April 2010, pp. 1-7.

Lombardi, E. 1994. Engineering Tests Performed on the X2000 and ICE High Speed Trainsets. In: *Proceedings of the 1994 ASME/IEEE Joint Rail Conference*, March 1994, Chicago, IL, pp. 13-21.

Lombardi, E., Warner, D.C., Sherrock, E.T., Whitten, B.T, and Jamieson, D.W. 2002. Field Testing and Monitoring of Rolling Stock under the new Federal Track and Equipment Regulations. In: *Proceedings of the 2002 ASME/IEEE Joint Rail Conference*, April 2002, Washington, DC, pp 161-169.

Lukaszewicz, P. 2006. Running Resistance – Results and Analysis of Full-Scale Test with Passenger and Freight Trains in Sweden. *Journal of Rail and Rapid Transit Part F: Proceedings of the Institution of Mechanical Engineers* 221: 183-193.

Marquis, B. 2011. Examination of Vehicle Performance at High Speed and High Cant Deficiency. In: *Proceedings of the ASME/ASCE/IEEE 2011 Joint Rail Conference, Pueblo, CO*, March 2011, pp. 1-9.

Martland, C.D. 2006. Improving On-Time Performance for Long-Distance Passenger Trains Operating on Freight Routes. *Journal of the Transportation Research Forum* 47: 63-80.

National Railroad Passenger Corporation (Amtrak). 2009a. *An Interim Assessment of Achieving Improved Trip Times on the Northeast Corridor*. National Railroad Passenger Corporation , Washington, D.C. URL: http://www.amtrak.com/ccurl/350/350/AmtrakNortheastCorridorAssessment,0.pdf. Accessed on 3/30/2013.

National Railroad Passenger Corporation (Amtrak). 2009b. *North Coast Hiawatha Passenger Rail Study*. National Railroad Passenger Corporation , Washington, D.C. URL: http://www.amtrak.com/ccurl/492/133/NorthCoastHiawathaServiceStudy.pdf. Accessed on 4/1/2013.

National Railroad Passenger Corporation (Amtrak). 2012. Hiawatha Service Schedule, National Railroad Passenger Corporation, Washington, D.C. URL: http://www.amtrak.com/ccurl/893/15/Hiawatha-Schedule-050712,0.pdf. Accessed on 5/5/2013.

Nelson, S. 2012. Railway Approval Needed for Lake Forest Stop. *Lake Forest-Lake Bluff Patch*, July 27, 2012.

New York Department of Transportation (NYDOT) . 2012. *High Speed Rail Empire Corridor Project, Online Briefing*. New York Department of Transportation . URL:

https://www.dot.ny.gov/content/delivery/Main-Projects/S93751-Home/S93751--Repository/ECHSR_Online_Briefing_March_2012.pdf. Accessed on 4/1/2013.

O'Dwyer, D. 1997. Realignment of Railway Curves. *Proceedings of the ICE – Transport* 124: 234-240.

Oregon Department of Transportation (ODOT). 2013. *New Talgo Trains Making their Way to , Oregon*. Oregon Department of Transportation, Salem, OR. URL: http://www.oregon.gov/ODOT/COMM/pages/talgohome.aspx. Accessed on 3/11/2013.

Perl, A. 2012. Assessing the recent reformulation of United States passenger rail policy. *Journal of Transport Geography* 22: 271-281.

Petersen, E.R., and Taylor, A.J. 2001. An Investment Planning Model for a New North-Central Railway in Brazil. *Journal of Transportation Research Part A: Policy and Practice* 35: 847-862.

Putallaz, Y., Rivier, R. 2004. Strategic Evolution of Railway Corridor Infrastructure: Dual Approach for Assessing Capacity Investments and M&R Strategies. *Computers in Railways* 9: 61-72.

Quandel Consultants LLC. 2011. *Cost Estimating Methodology for High Speed Rail on Shared Right-of-Way.* Minnesota Department of Transportation, St. Paul, MN. URL: http://www.dot.state.mn.us/passengerrail/mwrri/phase7.html. Accessed on 12/12/2012.

Rose, M.K. 2008. *Written Statement on the Findings of the National Surface Transportation Policy and Revenue Study Commission.* Hearing before the House Committee on Transportation and Infrastructure. URL: https://www.gpo.gov/fdsys/pkg/CHRG-110hhrg40625/pdf/CHRG-110hhrg40625.pdf . Accessed on 12/12/2017.

Smith, S.V. 1951. A New Train Performance Calculator. *Transactions of the American Institute of Electrical Engineers* 70: 657-660.

Sogin, S.L, Saat, M.R., and Barkan, C.P.L. 2011. Simulating the effects of higher speed passenger trains in single track freight networks. In: *Proceedings of the 2011 Winter Simulation Conference (WSC)* , Phoenix, USA, December 2011, 3679-3687.

Tang, H, Dick, C.T., Caughron, B.M., Feng, X, Wang, Q, and Barkan, C.P.L. 2016. *Transportation Research Record: Journal of the Transportation Research Board* 2546: 33-42.

Tsai, T., Jamieson, D.,Whitten, B., and Mee, B. 1995. Testing of Rail Vehicles for Higher Speeds on Existing Lines. In: *Proceedings of the IEEE/ASME Joint Rail Conference*, Baltimore, MD, April 1995, pp. 15-18.

Union Pacific Railroad (UPRR). 2012. *Union Pacific Higher Speed Passenger Rail Access Principles*. Union Pacific Railroad, Omaha, NE. Unpublished memorandum.

Union Pacific (UPRR). 2013. *Spiral Design Criteria.* Union Pacific Railroad, Omaha, NE.

Wardrop, A. W. 2009. Use of Railway Analysis Tools from an Australian Perspective. *Networks and Spatial Economics* 9: 123-143.

Wilson, S. 2010. Amtrak Cascades @ Pioneer Wa. Flickr Creative Commons Photo. URL: http://www.flickr.com/photos/locosteve/6765719655/. Accessed on 3/11/2013.

Wood, D., Robertson, S. 2002. Planning Tomorrow's Railway – Role of Technology in Infrastructure and Timetable Options Evaluation. *Computers in Railways* 8: 721-730.

Zarembski, A.M., and Resor R.R. 2004. *Technical Monograph: Estimating Maintenance Costs for Mixed High Speed Passenger and Freight Rail Corridors*. Federal Railroad Administration, Washington, D.C. Unpublished Report

APPENDIX A: PROJECT SELECTION MODEL PARAMETERS AND INPUT DATA

Table A.1: Model Indices

Indices	Name	Description
T	trains	index of different train services
N	segments	index of route segments
S	train speeds	index for train speed
C	infrastructure speed conditions	index for track speed condition

Table A.2: Model Variables

Variables	Name	Description
$v_{n,s,t}$	train speed	speed of train ‘t’ on segment ‘n’ (0,1)
$x_{n,c}$	track speed condition	condition of track on segment ‘n’ (0,1)
$z_{n,c,c^*,t}$	acceleration link variable	link variable used to indicate change in speed between segments ‘n’ (0,1)
$a_{n,t}$	segment acceleration distance	acceleration distance of train ‘t’ on segment ‘n’ (real number > 0)
$b_{n,t}$	segment braking distance	braking distance of train ‘t’ to segment ‘n’ (real number > 0)

Table A.3: Model Parameters

Parameters	Name	Description
$\theta_{n,t}$	train service importance factor	weight factor for different train services (real number > 0)
l_n	segment length	length of segment (real number >0)
δ_s	running time	running time at speed 's' per length (real number > 0)
$\tau_{s,s^*,t}$	acceleration braking delay time	acceleration and braking delay time from speed 's' to speed 's*' for train type 't' (real number > 0)
$p_{n,c}$	present value cost	present value capital construction and maintenance cost for condition 'c' on segment 'n' (real number > 0)
B	present value budget	budget of present value capital construction and maintenance costs (real number > 0)
σ_s	speed parameter (s)	speed parameter at speed condition 's' (real number > 0)
ν_c	speed parameter (c)	speed parameter at segment condition 'c' (real number > 0)
$h_{n,t}$	train service pattern	parameter which can set a maximum speed for train 't' on segment 'n' (real number > 0)
$\beta_{s,t}$	standard braking distance	cumulative braking distance from speed 's' for train type 't' (real number > 0)
$\alpha_{s,t}$	standard acceleration distance	cumulative acceleration distance to speed 's' for train type 't' (real number > 0)

Table A.4: Standard Train Consist Characteristics

Name	Weight (tons)	Power (hp)	Coefficient of Adhesion	Davis A Coef.	Davis Aa Coef.	Davis B Coef.	Cross Section Area (ft^2)	Davis CN Coef.	Number of Axles	Max Speed (MPH)
P42DC	134	4,250	0.25	1.5	18	0.03	110	10	4	110
Amfleet	58	-	-	1.5	18	0.03	110	3.5	4	125
Horizon	57	-	-	1.5	18	0.03	110	3.5	4	125
Horizon	57	-	-	1.5	18	0.03	110	3.5	4	125
Horizon	57	-	-	1.5	18	0.03	110	3.5	4	125
Horizon	57	-	-	1.5	18	0.03	110	3.5	4	125
Amfleet	58	-	-	1.5	18	0.03	110	3.5	4	125
P42DC	134	-	-	1.5	18	0.03	110	3.5	4	110

Table A.5: Standard Train Acceleration and Braking Time (minutes)

Starting Speed (MPH)	Ending Speed (MPH) 0	15	30	40	50	60	65	70	75	80	85	90	95	100	105	110
0	0.00	0.11	0.22	0.31	0.42	0.55	0.63	0.71	0.81	0.91	1.02	1.14	1.28	1.43	1.59	1.78
15	0.11	0.00	0.03	0.09	0.19	0.31	0.39	0.47	0.56	0.66	0.77	0.89	1.02	1.17	1.34	1.52
30	0.23	0.03	0.00	0.03	0.11	0.22	0.28	0.36	0.45	0.54	0.65	0.77	0.90	1.05	1.21	1.39
40	0.30	0.08	0.02	0.00	0.03	0.12	0.18	0.24	0.32	0.41	0.51	0.63	0.75	0.89	1.05	1.23
50	0.38	0.14	0.06	0.02	0.00	0.04	0.08	0.13	0.19	0.27	0.36	0.47	0.58	0.72	0.87	1.04
60	0.45	0.21	0.12	0.05	0.02	0.00	0.01	0.04	0.08	0.14	0.21	0.30	0.41	0.53	0.67	0.83
65	0.49	0.24	0.15	0.07	0.03	0.00	0.00	0.01	0.04	0.09	0.15	0.23	0.32	0.43	0.57	0.72
70	0.52	0.27	0.18	0.10	0.05	0.01	0.00	0.00	0.01	0.04	0.09	0.16	0.24	0.35	0.47	0.62
75	0.56	0.31	0.21	0.12	0.07	0.02	0.01	0.00	0.00	0.01	0.04	0.10	0.17	0.26	0.37	0.51
80	0.59	0.34	0.24	0.15	0.09	0.04	0.02	0.01	0.00	0.00	0.01	0.05	0.10	0.18	0.28	0.41
85	0.63	0.38	0.27	0.18	0.11	0.06	0.04	0.02	0.01	0.00	0.00	0.01	0.05	0.11	0.20	0.31
90	0.66	0.41	0.30	0.21	0.14	0.08	0.06	0.03	0.02	0.01	0.00	0.00	0.01	0.05	0.12	0.22
95	0.70	0.44	0.33	0.24	0.16	0.10	0.07	0.05	0.03	0.02	0.01	0.00	0.00	0.02	0.06	0.14
100	0.74	0.48	0.36	0.26	0.19	0.12	0.09	0.07	0.05	0.03	0.02	0.01	0.00	0.00	0.02	0.07
105	0.77	0.51	0.40	0.29	0.22	0.14	0.12	0.08	0.06	0.05	0.03	0.02	0.01	0.00	0.00	0.02
110	0.81	0.55	0.43	0.32	0.24	0.17	0.14	0.10	0.08	0.06	0.04	0.03	0.02	0.01	0.00	0.00

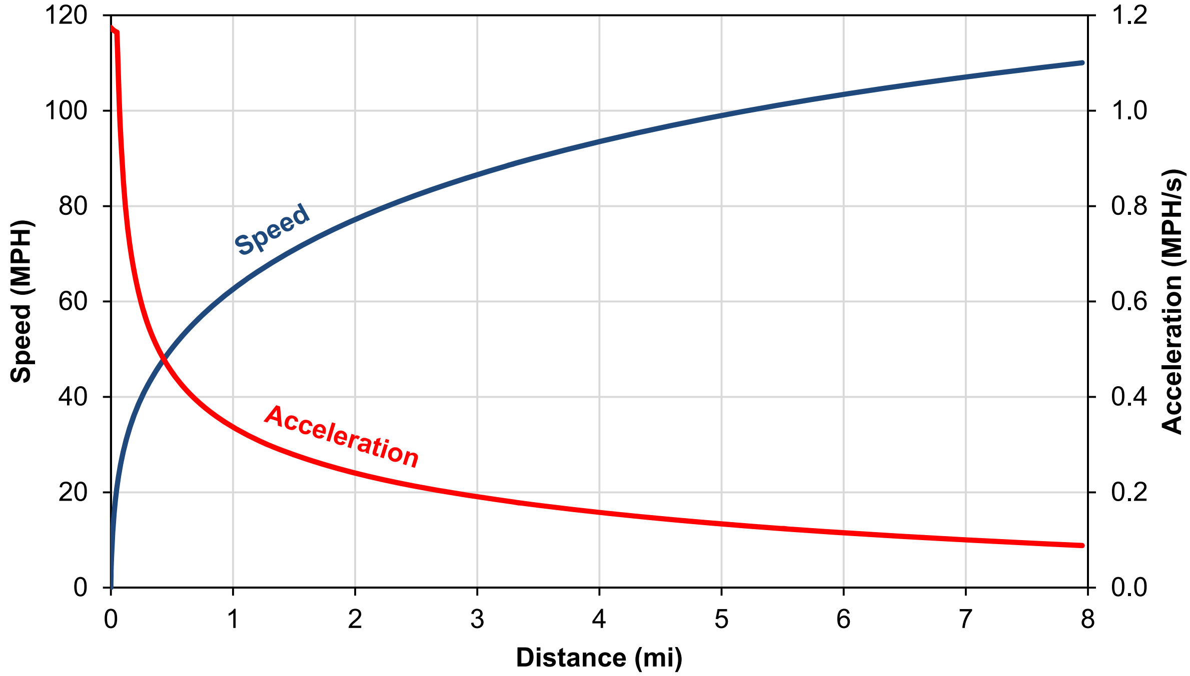


Figure A.1: Standard Train Consist Acceleration Profile

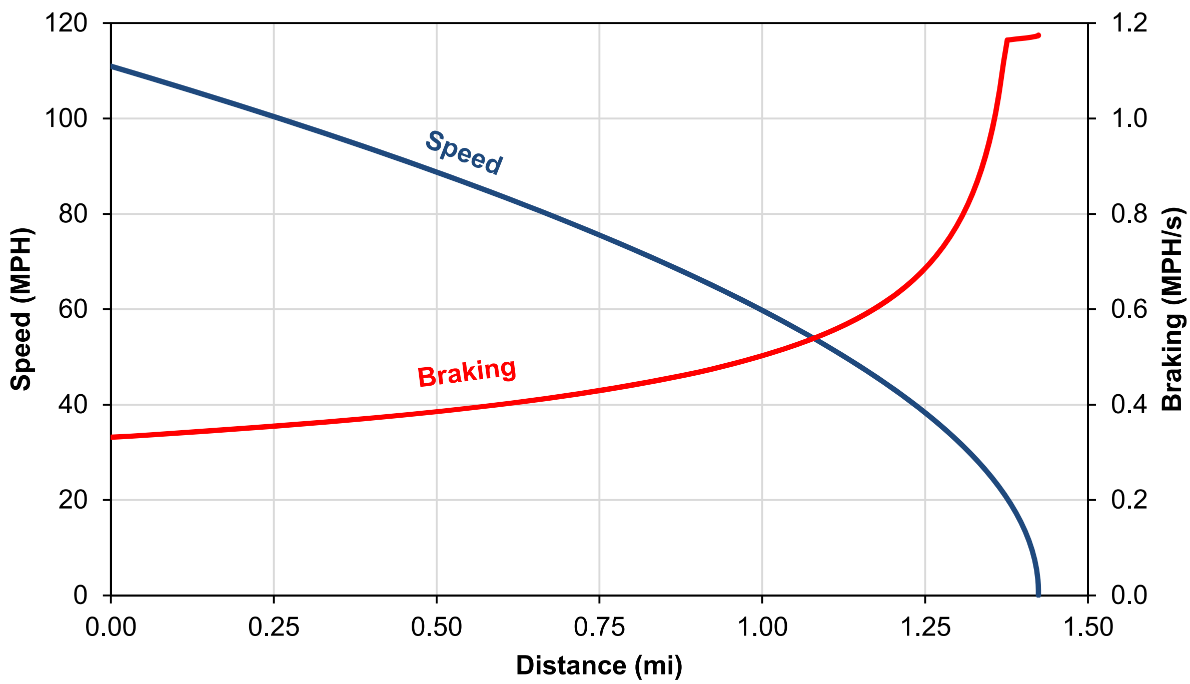


Figure A.2: Standard Train Consist Braking Profile

APPENDIX B: CURVE REALIGNMENT PARAMETERS

Table B.1: List of Parameters

Parameter	Description
A	Allowable track center shift at middle of curve body in feet
D	Degree if curve on 100 foot chord basis
D_o	Initial degree of curve
D_f	Final degree of curve
Δ	Spiral deflection angle in degrees
E_s	External distance at middle of curve body to curve PI
I	Deflection angle of entire curve in degrees
E_a	Actual superelevation in inches
$E_{u,f}$	Cant deficiency of freight traffic
$E_{u,p}$	Cant deficiency of passenger traffic
o	Offset distance from point of curve (P.C.) to tangent
R	Curve radius in feet
S	Spiral length in hundreds of feet
V_f	Speed of freight train in miles per hour
V_p	Speed of passenger train in miles per hour

APPENDIX C: SPREADSHEET BASED TPC INPUT DATA AND VBA CODE

Table C.1: Existing Amtrak *Hiawatha* Schedule (Amtrak 2012)

Northbound

Train Number ▸				329	331	333	335	337	339	341
Normal Days of Operation ▸				Mo-Sa	Daily	Daily	Daily	Daily	Daily	Daily
On Board Service ▸										
	Mile	Symbol								
Chicago, IL–Union Station (CT)	0	●♿QT	Dp	6 10A	8 25A	10 20A	1 05P	3 15P	5 08P	8 05P
Glenview, IL	18	●♿QT		6 32A	8 47A	10 42A	1 27P	3 37P	5 32P	8 27P
Sturtevant, WI (Racine)	63	○♿QT		7 10A	9 25A	11 20A	2 05P	4 15P	6 14P	9 05P
Milwaukee Airport Rail Sta., WI ✈	78	○♿QT 7		7 24A	9 39A	11 34A	2 19P	4 29P	6 28P	9 19P
Milwaukee, WI (CT) Oshkosh, Wausau—see back	86	●♿QT	Ar	7 39A	9 54A	11 49A	2 34P	4 44P	6 45P	9 34P

Southbound

Train Number ▸				330	332	334	336	338	340	342
Normal Days of Operation ▸				Mo-Sa	Daily	Daily	Daily	Daily	Daily	Daily
On Board Service ▸										
	Mile	Symbol								
Milwaukee, WI (CT) Oshkosh, Wausau—see back	0	●♿QT	Dp	6 15A	8 05A	11 00A	1 00P	3 00P	5 45P	7 35P
Milwaukee Airport Rail Sta., WI ✈	8	○♿QT 7		6 26A	8 15A	11 10A	1 10P	3 10P	5 55P	7 45P
Sturtevant, WI (Racine)	23	○♿QT		6 43A	8 28A	11 23A	1 23P	3 23P	6 08P	7 58P
Glenview, IL	68	●♿QT		7 25A	9 06A	12 01P	2 01P	4 01P	6 46P	8 36P
Chicago, IL–Union Station (CT)	86	●♿QT	Ar	7 57A	9 34A	12 29P	2 29P	4 29P	7 14P	9 04P

Table C.2: Diesel Trainset Performance Characteristics

Name	Wgt. (tons)	Length (ft)	Tract. Coef.	Power (hp)	Davis A Coef.	Davis Aa Coef.	Davis B Coef.	Cross Section Area (ft^2)	Davis CN Coef.	Number of Axles	Max Speed (MPH)	Starting TE (lbs)
P42DC	134	69	0.25	3,540	1.5	18	0.03	110	10	4	110	67,060
P42DC	134	69			1.5	18	0.03	110	3.5	4	110	
Amfleet	58	85			1.5	18	0.03	110	3.5	4	125	
Horizon	57	85			1.5	18	0.03	110	3.5	4	125	
Horizon	57	85			1.5	18	0.03	110	3.5	4	125	
Horizon	57	85			1.5	18	0.03	110	3.5	4	125	
Horizon	57	85			1.5	18	0.03	110	3.5	4	125	
Amfleet	58	85			1.5	18	0.03	110	3.5	4	125	

Table C.3: EMU Performance Characteristics

Name	Wgt. (tons)	Length (ft)	Tract. Coef.	Power (hp)	Davis A Coef.	Davis Aa Coef.	Davis B Coef.	Cross Section Area (ft^2)	Davis CN Coef.	Number of Axles	Max Speed (MPH)	Starting TE (lbs)
Valero-E	468	656	0.06	11,801	0.5	18	0.005	123.5	28	32	220	56,160

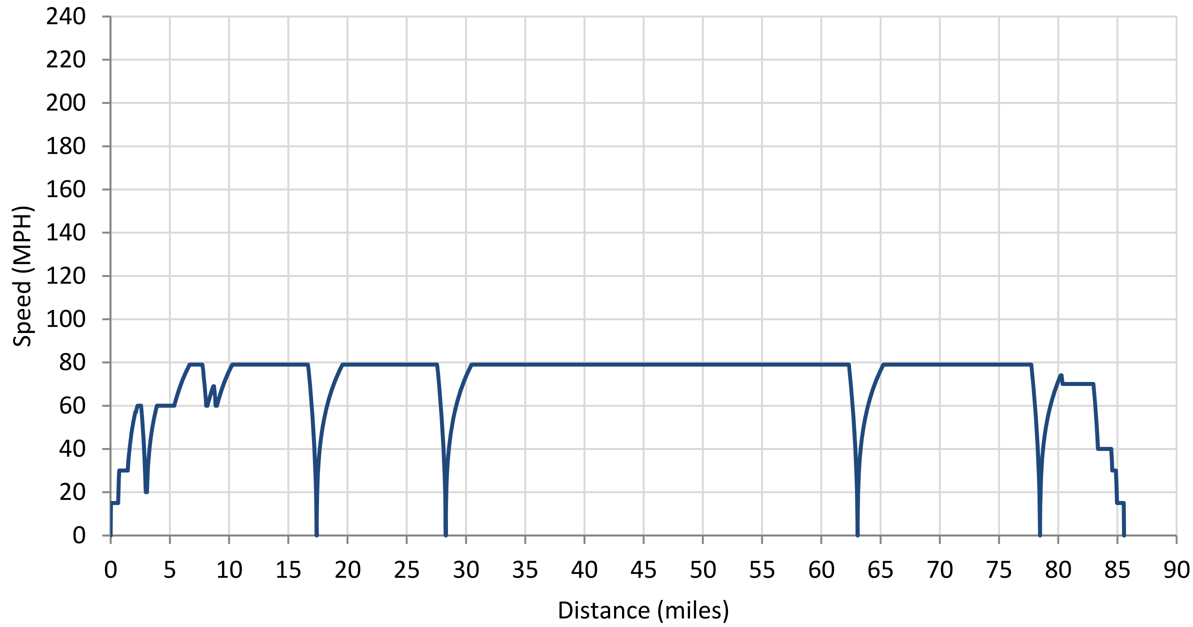


Figure C.1: Speed vs. Distance Profile D1

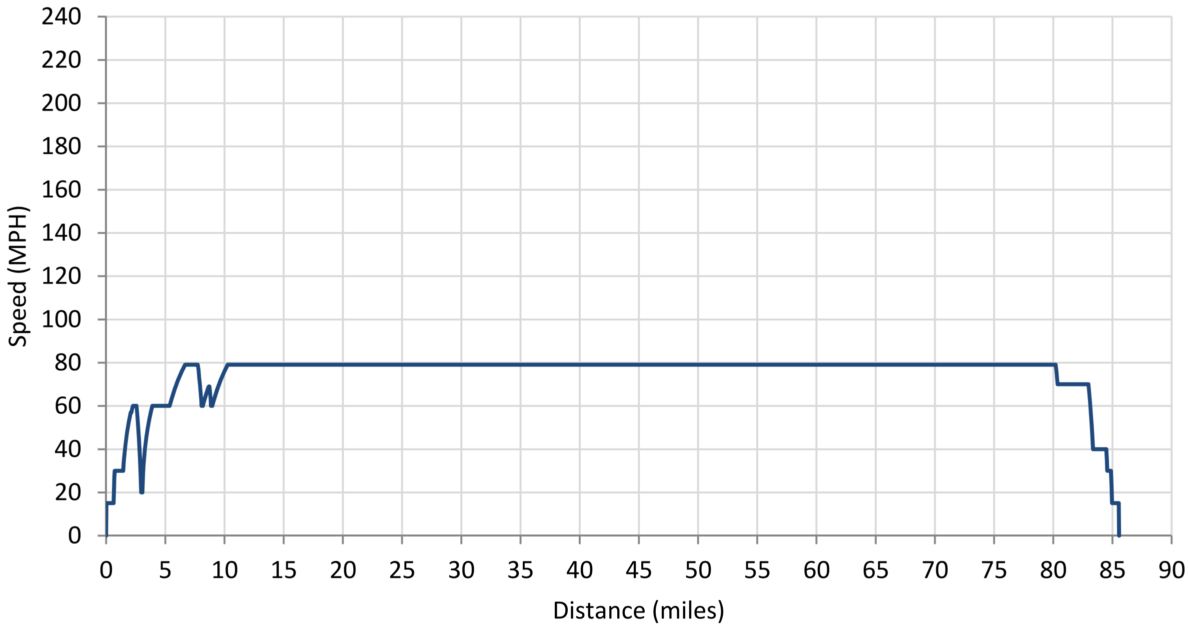


Figure C.2: Speed vs. Distance Profile D1-E

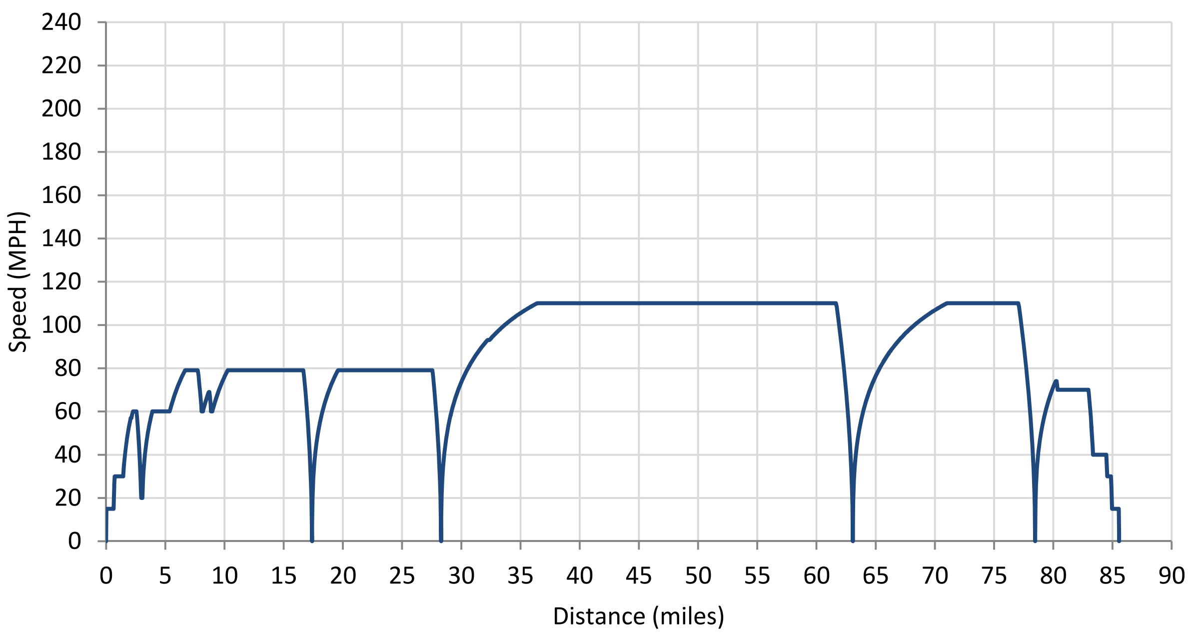


Figure C.3: Speed vs. Distance Profile D2

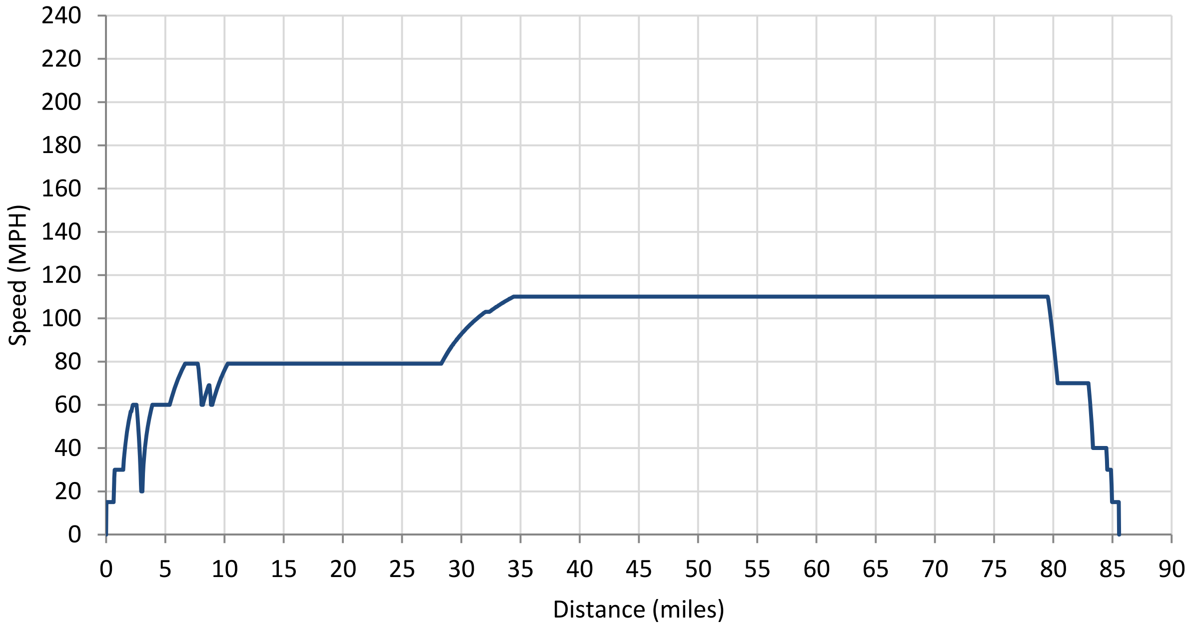


Figure C.4: Speed vs. Distance Profile D2-E

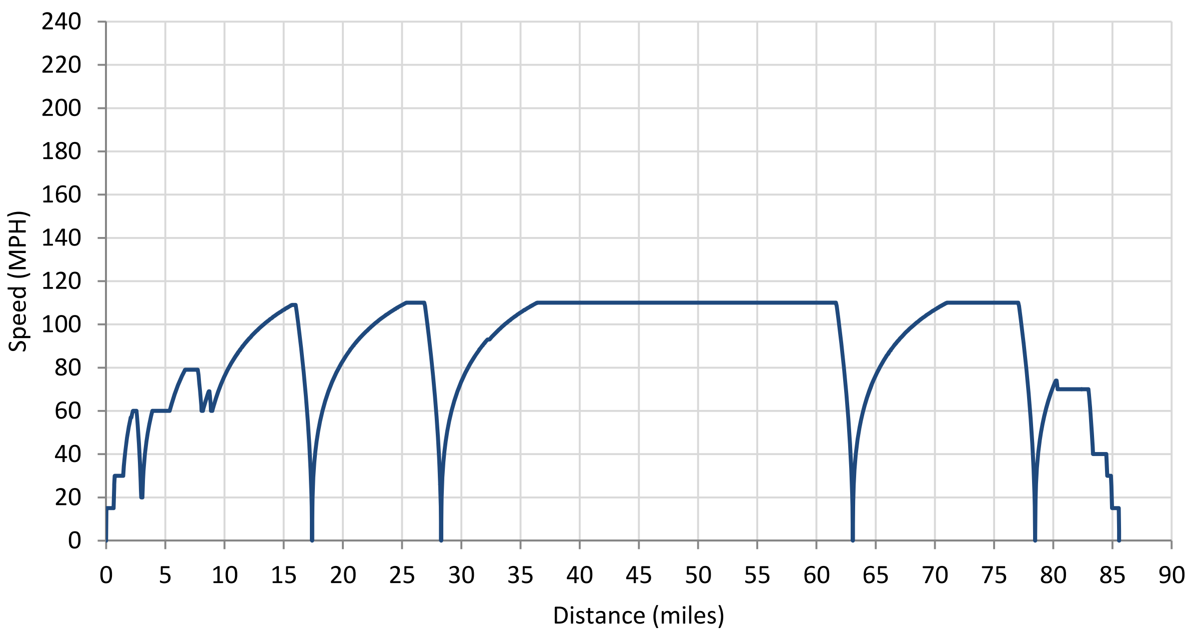


Figure C.5: Speed vs. Distance Profile D3

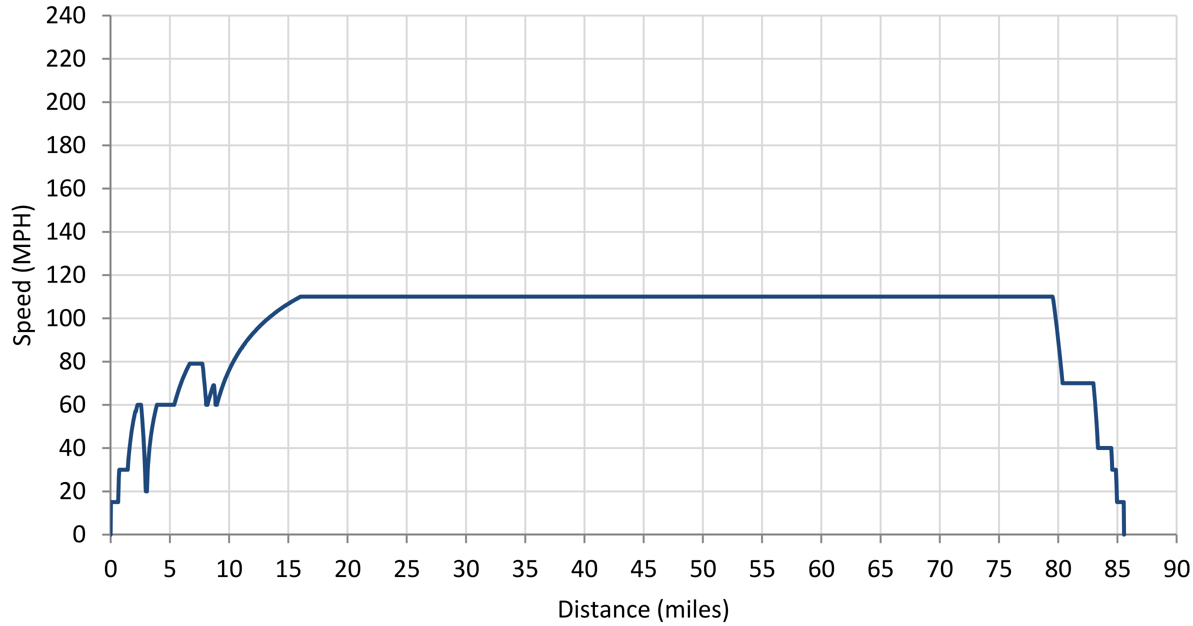


Figure C.6: Speed vs. Distance Profile D3-E

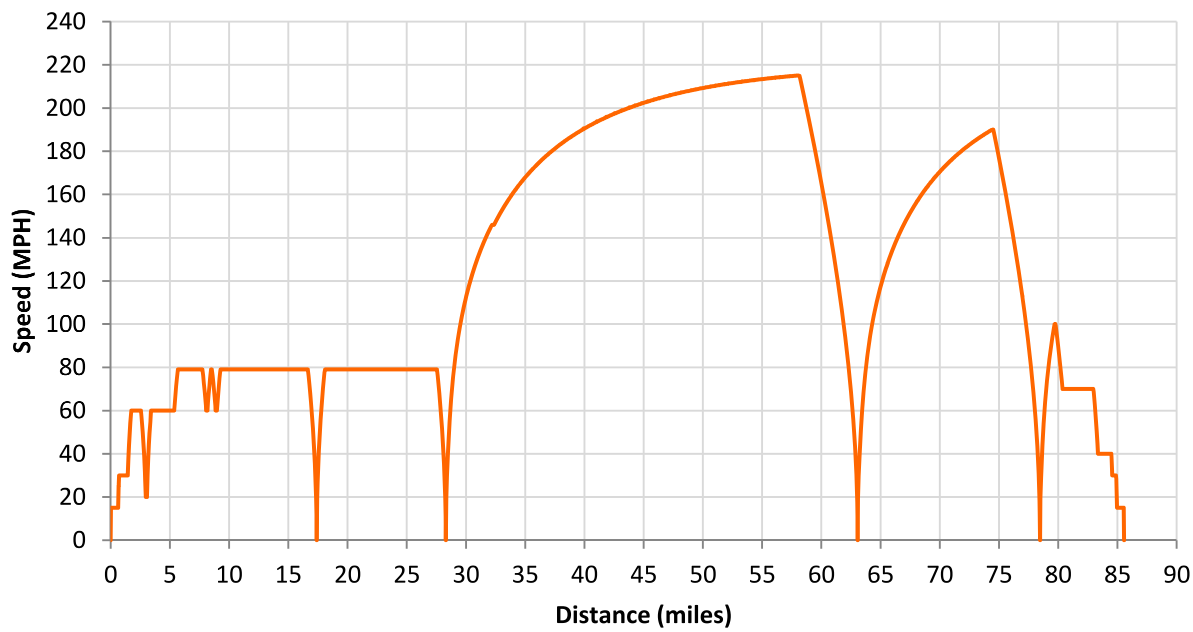


Figure C.7: Speed vs. Distance Profile E1

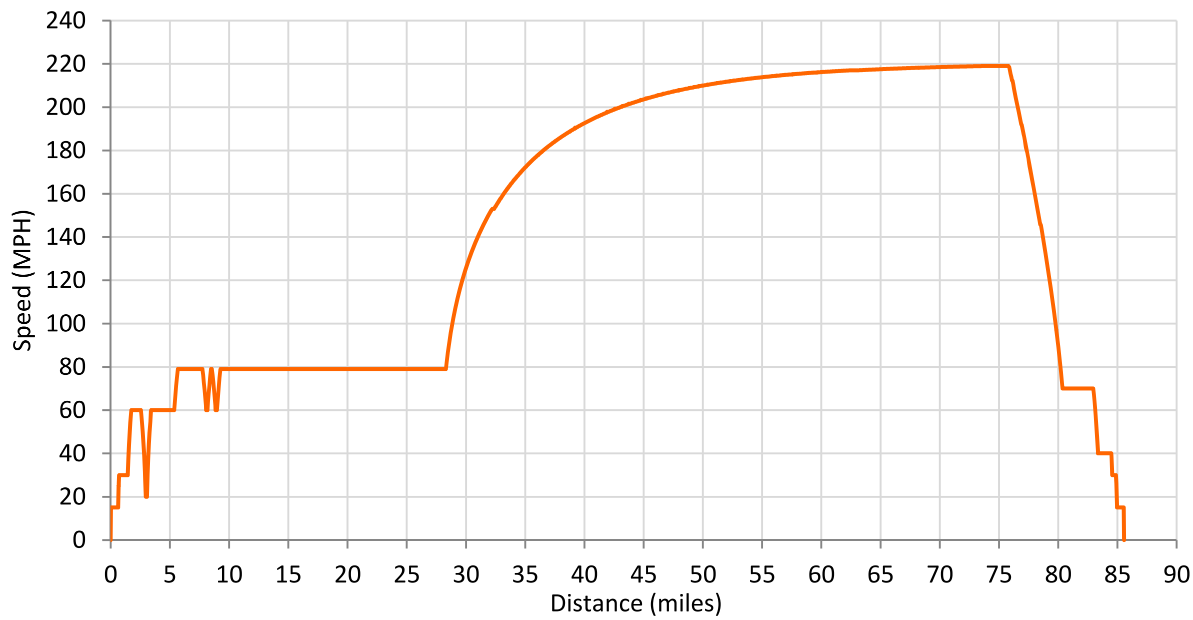


Figure C.8: Speed vs. Distance Profile E1-E

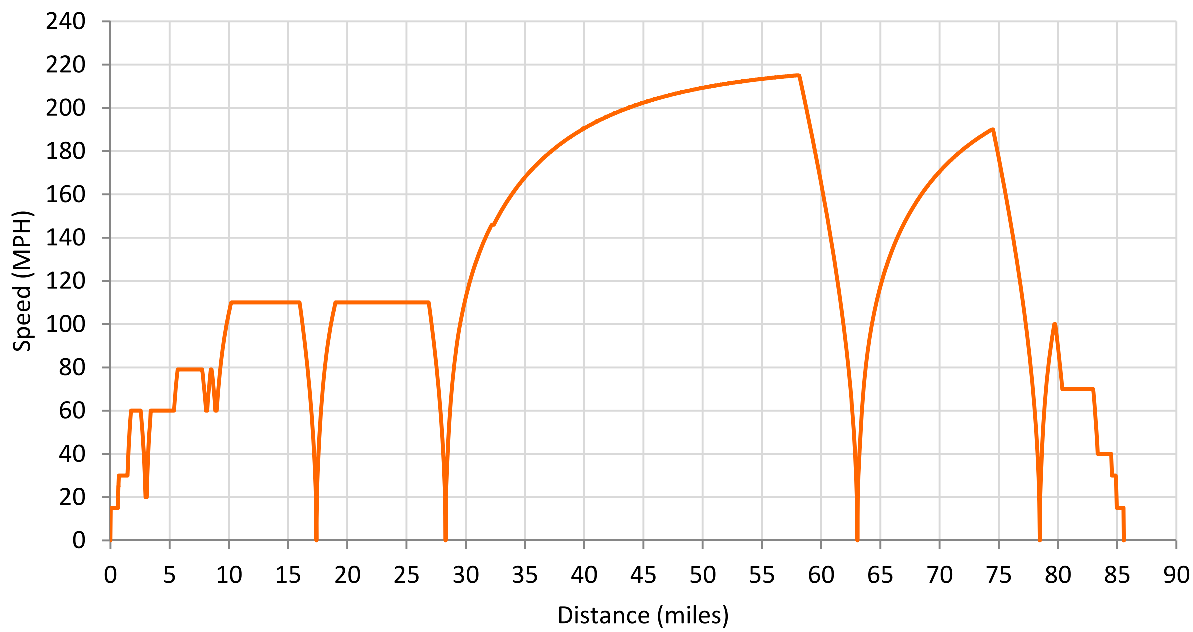


Figure C.9: Speed vs. Distance Profile E2

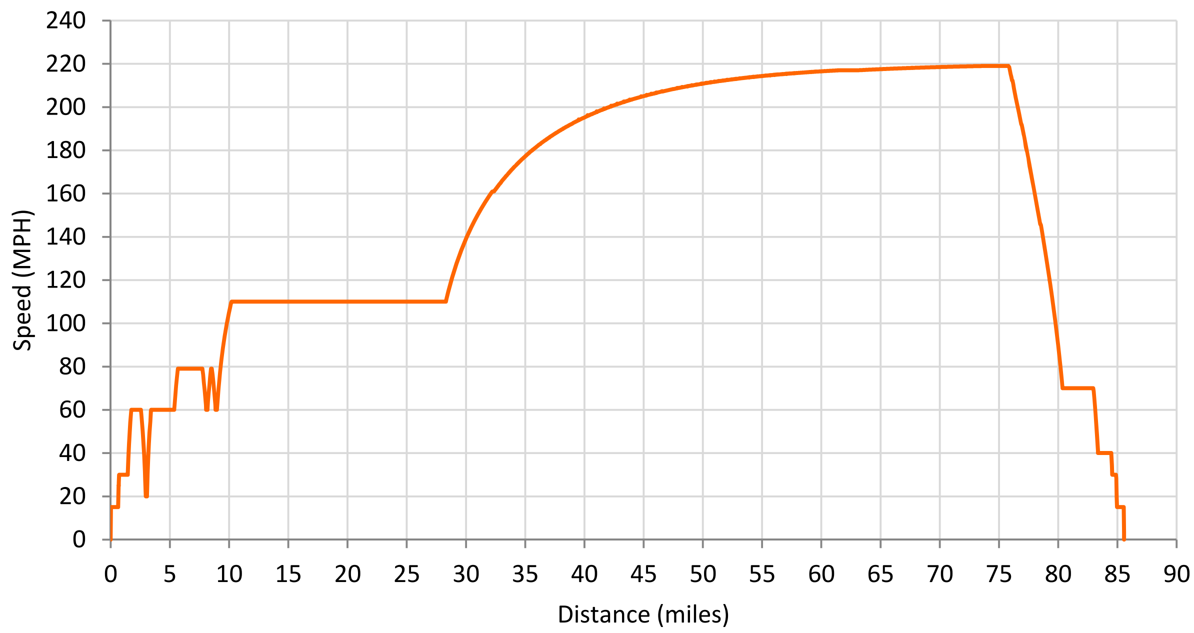


Figure C.10: Speed vs. Distance Profile E2-E

VBA code for Route Input Sheet

```
Option Explicit

Sub Route()

Sheets("Route").Activate

Dim r, temp1, quit As Integer

r = 1
quit = 0


'--------equivalent grade column-----------------------------------------

Range("I2", Range("I2").End(xlDown)).ClearContents

Do
```

```
r = r + 1

If Cells(r, 6) = "" Then
   Cells(r, 6) = 0
End If

If Cells(r, 7) = "" Then
   Cells(r, 7) = 0
End If

Cells(r, 9) = Cells(r, 6) + 0.04 * Cells(r, 7)

If r = 2 Then
   Cells(r, 4) = 0
Else
   Cells(r, 4) = Cells(r - 1, 4) + Cells(r - 1, 3)
End If



If Cells(r + 1, 3) = "" Then
   quit = 1
   End If

Loop Until quit > 0

'------------------------------------------------------------------------

End Sub
```

VBA code for Rail Vehicle Input Sheet

```
Option Explicit

Sub Vehicle()

Sheets("Rail Vehicle").Activate

Dim r, temp1, quit As Integer
Dim mass, tract, resist, speed As Double

r = 1
quit = 0

'--------vehicle totals calculation------------------------------------------

Do

r = r + 1

Cells(r, 13).ClearContents
Cells(r, 13) = Cells(r, 2) * (Cells(r, 6) + (Cells(r, 7) * Cells(r, 11) / Cells(r, 2)))

Cells(r, 14).ClearContents
Cells(r, 14) = Cells(r, 8) * Cells(r, 2)

Cells(r, 15).ClearContents
Cells(r, 15) = Cells(r, 9) * Cells(r, 10) / (10000)

Cells(r, 16).ClearContents
Cells(r, 16) = Cells(r, 2) * 2000 * Cells(r, 4)

If Cells(r + 1, 1) = "" Then
   quit = 1
   End If
```

```
Loop Until quit > 0

'--------generate TE, resistance, acceleration plots-----------------------------------------

Range("T3", Range("W3").End(xlDown)).ClearContents



If Cells(18, 18) = 1 Then

speed = 0
mass = Cells(7, 18)

Do

If speed = 0 Then
   tract = Sheets("Rail Vehicle").Cells(8, 18)
Else
   If (Cells(9, 18) * 375 * Cells(17, 18) / speed) > Cells(8, 18) Then
      tract = Cells(8, 18)
   Else
      tract = (Cells(9, 18) * 375 * Cells(17, 18) / speed)
   End If
End If

resist = Cells(10, 18) + (speed * Cells(11, 18)) + ((speed ^ 2) * Cells(12, 18))

Cells(speed + 3, 20) = speed
Cells(speed + 3, 21) = tract
Cells(speed + 3, 22) = resist
Cells(speed + 3, 23) = (3600 / 5280) * (tract - resist) / mass

speed = speed + 1

Loop Until speed > 220
```

```
End If

End Sub
```

VBA code for TPC Calculator Sheet

```
Option Explicit

Sub TPC()

Application.ScreenUpdating = False

'Sheets("Calculator").Activate

Dim r, rmax, plotrow, gmax, c, quit, quit2, t, ti, pr1, pr2 As Integer
Dim temp1 As Variant
Dim speed, X1, X2, length, x, xmax, gtot, glookup, V1, V2, mass, tract, resist, weight, prdist, prdist2, prminspd, prdistmin As Double

r = 1
rmax = 1
plotrow = 1
gmax = 1
c = 1
quit = 0
t = 0
V1 = 0
V2 = 0
X1 = 0
X2 = 0
x = 0
gtot = 0
xmax = 0
ti = 1
glookup = 0
length = Sheets("Rail Vehicle").Cells(3, 18)
mass = 1
weight = 1
```

```
speed = 1
tract = 1
resist = 1
quit2 = 0

Sheets("Sub3").Cells.Clear

'-------copy route input data----------------------------------------------

Range("A2", Range("R2").End(xlDown)).ClearContents

Do
Do

r = r + 1

temp1 = Sheets("Route").Cells(r, c)

If Sheets("Route").Cells(r + 1, 3) = "" Then
    quit = 1
End If

rmax = r
xmax = Sheets("Route").Cells(r, 4)

Loop Until quit > 0
c = c + 1
r = 1
quit = 0
Loop Until c > 18

r = 1
rmax = 1
gmax = 1
c = 1
```

```
quit = 0

Do
Do

r = r + 1

temp1 = Sheets("Route").Cells(r, c)

If Sheets("Route").Cells(r + 1, 3) = "" Then
    quit = 1
End If

Cells(r, c) = temp1

rmax = r
xmax = Sheets("Route").Cells(r, 4)

Loop Until quit > 0
c = c + 1
r = 1
quit = 0
Loop Until c > 9

'--------initial adjusted speed with vehicle considerations-----------------

r = 1
quit = 0

Do

r = r + 1

If Sheets("Calculator").Cells(r, 5) < Sheets("Rail Vehicle").Cells(4, 18) Then
   Sheets("Calculator").Cells(r, 10) = Sheets("Calculator").Cells(r, 5)
```

```
Else
   Sheets("Calculator").Cells(r, 10) = Sheets("Rail Vehicle").Cells(4, 18)
End If

Loop Until r = rmax

'--------consist specific equivalent grade calculation---------------------

Sheets("Sub2").Cells.Clear

   Do
      x = x + 1

      If x = 1 Then
         Sheets("Sub2").Cells(x, 1) = 0
      Else
         Sheets("Sub2").Cells(x, 1) = Sheets("Sub2").Cells(x - 1, 1) + Sheets("Rail Vehicle").Cells(13, 18)
      End If

      gtot = 0
      ti = 1

      Do
         glookup = Sheets("Sub2").Cells(x, 1) + ((ti - 1) * length / (Sheets("Rail Vehicle").Cells(14, 18) - 1))
         gtot = gtot + Sheets("Route").Application.VLookup(glookup, Sheets("Route").Range("D2", "I" & rmax), 6, True)
         ti = ti + 1
      Loop Until ti > Sheets("Rail Vehicle").Cells(14, 18)

      Sheets("Sub2").Cells(x, 2) = gtot / (ti - 1)
      Sheets("Sub2").Cells(x, 3) = Sheets("Route").Application.VLookup(Sheets("Sub2").Cells(x, 1), Sheets("Route").Range("D2", "I" & rmax),
6, True)

      gmax = x

   Loop Until Sheets("Sub2").Cells(x, 1) > xmax
```

```
'--------acceleration and deceleration calculation-------------------------
Do

r = 1
quit = 0
quit2 = 0

Do

r = r + 1

If r = 2 Then
    Sheets("Calculator").Cells(r, 11) = 0
    Sheets("Calculator").Cells(r, 14) = 0
Else

'-------------acceleration--------------------------------------------------------------------------------
    If Sheets("Calculator").Cells(r, 10) > Sheets("Calculator").Cells(r - 1, 10) Then

        V2 = Sheets("Calculator").Cells(r, 10)
        V1 = Sheets("Calculator").Cells(r - 1, 10)
        X1 = Sheets("Calculator").Cells(r, 4)
        X2 = Sheets("Calculator").Cells(r + 1, 4)
        t = 1
        quit = 0
        mass = Sheets("Rail Vehicle").Cells(7, 18)
        weight = Sheets("Rail Vehicle").Cells(6, 18)

        Sheets("Sub").Cells.Clear

        Do
            Sheets("Sub").Cells(t, 1) = t
            If t = 1 Then
                Sheets("Sub").Cells(t, 2) = X1
```

```
        Sheets("Sub").Cells(t, 3) = V1
        Sheets("Sub").Cells(t, 6) = 0
    Else
        Sheets("Sub").Cells(t, 2) = Sheets("Sub").Cells(t - 1, 2) + (Sheets("Sub").Cells(t - 1, 3) / 3600)
        Sheets("Sub").Cells(t, 6) = Sheets("Sub").Cells(t - 1, 6) + (Sheets("Sub").Cells(t - 1, 3) / 3600)
        Sheets("Sub").Cells(t, 3) = Sheets("Sub").Cells(t - 1, 3) + (Sheets("Sub").Cells(t - 1, 5))
    End If

    '--- copy data for plot

        Sheets("Sub3").Cells(plotrow, 2) = Sheets("Sub").Cells(t, 2)
        Sheets("Sub3").Cells(plotrow, 3) = Sheets("Sub").Cells(t, 3)
        plotrow = plotrow + 1

    '--- copy data for plot


    speed = Sheets("Sub").Cells(t, 3)

    If speed = 0 Then
        tract = Sheets("Rail Vehicle").Cells(8, 18)
    Else
        If (Sheets("Rail Vehicle").Cells(9, 18) * 375 * Sheets("Rail Vehicle").Cells(17, 18) / speed) > Sheets("Rail Vehicle").Cells(8, 18) Then
            tract = Sheets("Rail Vehicle").Cells(8, 18)
        Else
            tract = (Sheets("Rail Vehicle").Cells(9, 18) * 375 * Sheets("Rail Vehicle").Cells(17, 18) / speed)
        End If
    End If

    resist = Sheets("Rail Vehicle").Cells(10, 18) + (speed * Sheets("Rail Vehicle").Cells(11, 18)) + ((speed ^ 2) * Sheets("Rail
Vehicle").Cells(12, 18)) + (Sheets("Sub").Cells(t, 4) * 20 * Sheets("Rail Vehicle").Cells(5, 18))

    Sheets("Sub").Cells(t, 4) = Sheets("Sub2").Application.VLookup(Sheets("Sub").Cells(t, 2), Sheets("Sub2").Range("A1", "C" & gmax), 2,
True)
```

```
            '--added grade resistance

            resist = resist + (weight * (Sheets("Sub").Cells(t, 4) / 100))

            '--added grade resistance

            Sheets("Sub").Cells(t, 5) = (3600 / 5280) * (tract - resist) / mass

            If Sheets("Sub").Cells(t, 3) > V2 Then
                quit = 1
            End If

            t = t + 1

        Loop Until quit = 1

        Sheets("Calculator").Cells(r, 11) = Sheets("Sub").Cells(t - 1, 6)
        Sheets("Calculator").Cells(r, 14) = (t - 1) / 60
        '-------------
    Else
        Sheets("Calculator").Cells(r, 11) = 0
        Sheets("Calculator").Cells(r, 14) = 0
    End If
End If

'-------------deceleration---------------------------------------------------------------------------------

If Sheets("Calculator").Cells(r, 10) > Sheets("Calculator").Cells(r + 1, 10) Then

        V2 = Sheets("Calculator").Cells(r + 1, 10)
        V1 = Sheets("Calculator").Cells(r, 10)
        X1 = Sheets("Calculator").Cells(r, 4)
        X2 = Sheets("Calculator").Cells(r + 1, 4)
        t = 1
        quit = 0
```

```
mass = Sheets("Rail Vehicle").Cells(7, 18)

Sheets("Sub").Cells.Clear

Do
    Sheets("Sub").Cells(t, 1) = t
    If t = 1 Then
        Sheets("Sub").Cells(t, 2) = X2
        Sheets("Sub").Cells(t, 3) = V2
        Sheets("Sub").Cells(t, 6) = 0
    Else
        Sheets("Sub").Cells(t, 2) = Sheets("Sub").Cells(t - 1, 2) - (Sheets("Sub").Cells(t - 1, 3) / 3600)
        Sheets("Sub").Cells(t, 6) = Sheets("Sub").Cells(t - 1, 6) + (Sheets("Sub").Cells(t - 1, 3) / 3600)
        Sheets("Sub").Cells(t, 3) = Sheets("Sub").Cells(t - 1, 3) + (Sheets("Sub").Cells(t - 1, 5))
    End If

    '--- copy data for plot

        Sheets("Sub3").Cells(plotrow, 2) = Sheets("Sub").Cells(t, 2)
        Sheets("Sub3").Cells(plotrow, 3) = Sheets("Sub").Cells(t, 3)
        plotrow = plotrow + 1

    '--- copy data for plot

    speed = Sheets("Sub").Cells(t, 3)

    If speed = 0 Then
        tract = Sheets("Rail Vehicle").Cells(8, 18)
    Else
        If (Sheets("Rail Vehicle").Cells(9, 18) * 375 * Sheets("Rail Vehicle").Cells(17, 18) / speed) > Sheets("Rail Vehicle").Cells(8, 18) Then
            tract = Sheets("Rail Vehicle").Cells(8, 18)
        Else
            tract = (Sheets("Rail Vehicle").Cells(9, 18) * 375 * Sheets("Rail Vehicle").Cells(17, 18) / speed)
        End If
    End If
```

```
            resist = Sheets("Rail Vehicle").Cells(10, 18) + (speed * Sheets("Rail Vehicle").Cells(11, 18)) + ((speed ^ 2) * Sheets("Rail Vehicle").Cells(12, 18)) + (Sheets("Sub").Cells(t, 4) * 20 * Sheets("Rail Vehicle").Cells(5, 18))

            Sheets("Sub").Cells(t, 4) = Sheets("Sub2").Application.VLookup(Sheets("Sub").Cells(t, 2), Sheets("Sub2").Range("A1", "C" & gmax), 2, True)

            '--added grade resistance

            resist = resist + (weight * (Sheets("Sub").Cells(t, 4) / 100))

            '--added grade resistance

            Sheets("Sub").Cells(t, 5) = (3600 / 5280) * (resist + (0.05 * weight)) / mass
            '*******temporary braking function needs refinement*******
            If Sheets("Sub").Cells(t, 3) > V1 Then
               quit = 1
            End If

            t = t + 1

        Loop Until quit = 1

        Sheets("Calculator").Cells(r, 12) = Sheets("Sub").Cells(t - 1, 6)
        Sheets("Calculator").Cells(r, 15) = (t - 1) / 60

Else
    Sheets("Calculator").Cells(r, 12) = 0
    Sheets("Calculator").Cells(r, 15) = 0
End If

'-------------cruise distance--------------------

Sheets("Calculator").Cells(r, 13) = Sheets("Calculator").Cells(r, 3) - Sheets("Calculator").Cells(r, 11) - Sheets("Calculator").Cells(r, 12)
```

```
If Sheets("Calculator").Cells(r, 13) < 0 Then
    quit2 = quit2 + 1
    Sheets("Calculator").Cells(r, 10) = Sheets("Calculator").Cells(r, 10) - 1
End If

Loop Until r = rmax

Loop Until quit2 < 1

'--------check feasibility of actual speeds---------------------------------

'Sheets("Sub").Cells.Clear
'Sheets("Sub2").Cells.Clear

'--------total and cumulative time calculation------------------------------

r = 1

Do

r = r + 1

If Sheets("Calculator").Cells(r, 10) = 0 Then
    Sheets("Calculator").Cells(r, 16) = 0
    Else
    Sheets("Calculator").Cells(r, 16) = (Sheets("Calculator").Cells(r, 13) / Sheets("Calculator").Cells(r, 10)) * 60
    End If

Sheets("Calculator").Cells(r, 17) = Sheets("Calculator").Cells(r, 16) + Sheets("Calculator").Cells(r, 15) + Sheets("Calculator").Cells(r, 14) +
Sheets("Calculator").Cells(r, 8)

If r = 2 Then
    Sheets("Calculator").Cells(r, 18) = 0
Else
    Sheets("Calculator").Cells(r, 18) = Sheets("Calculator").Cells(r - 1, 18) + Sheets("Calculator").Cells(r - 1, 17)
```

```
End If

Loop Until r = rmax

'--------process plot data-------------------------------------------------

Sheets("Sub3").Range("B:C").Sort Key1:=Sheets("Sub3").Range("B1"), _
    Order1:=xlAscending, Header:=xlNo, OrderCustom:=1, MatchCase:=False, _
    Orientation:=xlTopToBottom, DataOption1:=xlSortNormal

'------filter plot data----------------------------------------------------

pr1 = 1
pr2 = 2

Do

    If pr1 = 1 Then
        prdist2 = Sheets("Sub3").Cells(pr1, 2)
    End If

    prdist = Sheets("Sub3").Cells(pr1, 2)

    If Sheets("Sub3").Cells(pr1, 3) < prminspd Then
        prminspd = Sheets("Sub3").Cells(pr1, 3)
        prdistmin = Sheets("Sub3").Cells(pr1, 2)
    End If

    If prdist > prdist2 + 0.05 Then
        Sheets("Sub3").Cells(pr2, 5) = prdistmin

        If prminspd > Application.WorksheetFunction.VLookup(prdistmin, Range("D2:J100"), 7, True) Then

            prminspd = Application.WorksheetFunction.VLookup(prdistmin, Range("D2:J100"), 7, True)
```

```
        End If

        Sheets("Sub3").Cells(pr2, 6) = prminspd
        prdist2 = prdist
        prminspd = 250
        prdistmin = 0
        pr2 = pr2 + 1
    End If

    pr1 = pr1 + 1

Loop Until Sheets("Sub3").Cells(pr1, 2) = ""

'-------add additional plot points from calculator sheet--------------------

r = 2

Do

If Sheets("Calculator").Cells(r, 11) > 0 Then

    Sheets("Sub3").Cells(pr2, 5) = Sheets("Calculator").Cells(r, 4)
    Sheets("Sub3").Cells(pr2, 6) = Sheets("Calculator").Cells(r - 1, 10)

    pr2 = pr2 + 1

    Sheets("Sub3").Cells(pr2, 5) = Sheets("Calculator").Cells(r, 4) + Sheets("Calculator").Cells(r, 11)
    Sheets("Sub3").Cells(pr2, 6) = Sheets("Calculator").Cells(r, 10)

    pr2 = pr2 + 1

End If

If Sheets("Calculator").Cells(r, 12) > 0 Then
```

```
    Sheets("Sub3").Cells(pr2, 5) = Sheets("Calculator").Cells(r + 1, 4)
    Sheets("Sub3").Cells(pr2, 6) = Sheets("Calculator").Cells(r + 1, 10)

    pr2 = pr2 + 1

    Sheets("Sub3").Cells(pr2, 5) = Sheets("Calculator").Cells(r + 1, 4) - Sheets("Calculator").Cells(r, 12)
    Sheets("Sub3").Cells(pr2, 6) = Sheets("Calculator").Cells(r, 10)

    pr2 = pr2 + 1

End If

r = r + 1

Loop Until r = rmax


'--------sort plot data-----------------------------------------------------

Sheets("Sub3").Range("E:F").Sort Key1:=Sheets("Sub3").Range("E1"), _
    Order1:=xlAscending, Header:=xlNo, OrderCustom:=1, MatchCase:=False, _
    Orientation:=xlTopToBottom, DataOption1:=xlSortNormal

'--------------------------------------------------------------------------

Application.ScreenUpdating = True

End Sub
```

VBA code for Speed vs. Distance Plot Update

```
Option Explicit

Sub UpdatePlot()

Columns("A:B").ClearContents

Dim r As Integer

r = 0

Do

r = r + 1

Cells(r, 1) = Sheets("Sub3").Cells(r, 5)
Cells(r, 2) = Sheets("Sub3").Cells(r, 6)

Loop Until Sheets("Sub3").Cells(r, 5) >= Application.WorksheetFunction.Max(Sheets("Route").Range("D:D"))

End Sub
```